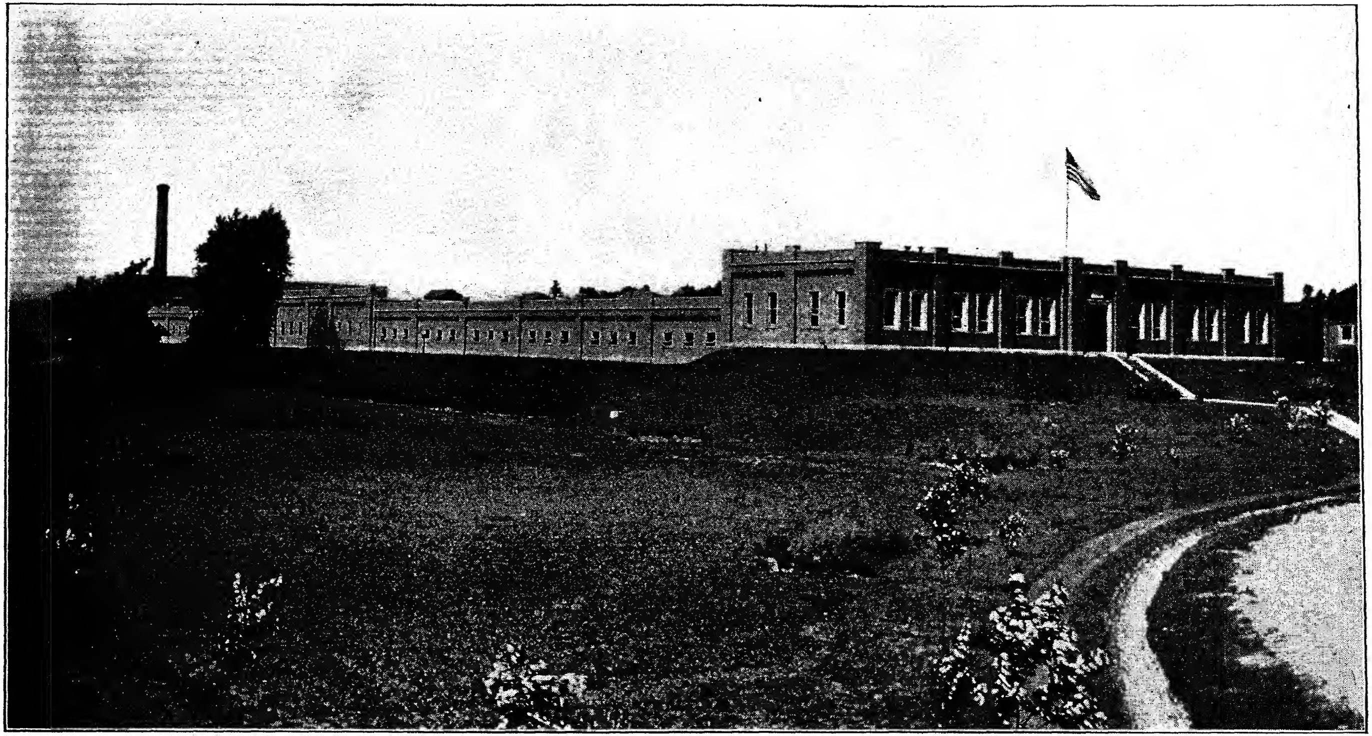
WATER PURIFICATION



CINCINNATI FILTRATION PLANT.

(*Frontispiece.*)

WATER PURIFICATION

BY

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PREFACE

In writing this book the object has been to provide the reader with a fairly complete account of the development of the art of water purification. As a knowledge of the physical, chemical and biological characteristics of natural waters is a prerequisite to the proper understanding of purification processes, a consideration of the properties of various classes of waters has been thought advisable. The relation of polluted public water supplies to water-borne diseases has received especial attention because of its importance. The various steps in purification processes, such as plain sedimentation, coagulation, filtration and disinfection, are described in considerable detail. Special chapters are devoted to water softening, and to the removal of iron and manganese from ground water supplies.

The rapid progress made in the art of clarifying and purifying turbid waters in the United States during the past quarter of a century, has been notable. The evolution of the rapid sand filter from its crude beginnings to its present well-developed state for purifying waters of this type, is distinctly the result of research work undertaken during the early part of this period. It was the author's good fortune to have been identified with some of the earlier investigations of this problem, and to have been able to follow its solution closely in actual practice throughout the whole period.

The experiences of other investigators, as disclosed by their published papers, have been drawn upon freely, and while no exhaustive examination of the extensive literature of the subject has been attempted, it is believed that the writers quoted and referred to are sufficiently representative to provide the reader with ample information upon the subjects discussed.

The writer is indebted to a number of his friends who have kindly supplied him with original information, illustrations, or assistance in writing technical descriptions of apparatus for some parts of the book; and to manufacturers of special devices

used in filter plant construction, who have loaned original drawings and photographs for reproduction.

The author desires to acknowledge especially the contribution of the subject matter in the appendices written by Mr. C. N. Miller, Assoc. M. Am. Soc. C. E., dealing with the hydraulics of the flow of water through filters, and with the discharge of water from waste-water troughs in the operation of rapid sand filters. The author is also under special obligation to Mr. S. J. Hauser, Chemist and Bacteriologist of the Cincinnati Water Purification Plant, for his kindly assistance in reading the proof of the book and in preparing the index.

CINCINNATI, OHIO,
March 1, 1917.

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WATER PURIFICATION

CHAPTER I

INTRODUCTION

Water purification may be broadly defined as the art of removing foreign and polluting substances from solution and suspension in water. The character and amount of these impurities, and the object for which their removal is undertaken, naturally leads to the treatment of this subject under the following heads:

1. Rendering impure water potable and hygienically safe for drinking purposes, and suitable for industrial uses.

2. Removing putrescible organic matter and disease-producing organisms from the fouled water supply or sewage of a community.

In the following pages it is intended to treat only of the first division of this subject, since the second covers too broad a field to bring it within the scope of this volume.

Both the kind and the amount of impurities in a polluted water affect the methods which can be employed for its purification, and this is so peculiarly true of sewage that its treatment has become an art in itself. Those waters which are to be purified for drinking purposes demand that a degree of purity shall be produced which is not possible and usually unnecessary in the case of sewage. The purification of the latter is, as a rule, quite incomplete as compared with that required for a safe drinking water, and obviously the object to be attained is quite different. In the case of waters to be treated for making them suitable for steam-boiler purposes, or for general industrial uses, a slightly different object is sought. The softening of hard waters or the removal of compounds of iron or manganese may be an integral part of the purification of a public drinking water supply, in which case it is combined with those processes utilized for making a water hygienically safe. Of course it is quite possible to carry on purification for industrial purposes independently of the more refined methods used for drinking water, and frequently this course is pursued in connection with manufacturing plants.

HISTORICAL

Ancient Systems of Water Supply.—Good water for drinking purposes has doubtless been appreciated by the human race from time immemorial, for it does not take a very high degree of intelligence to discriminate between a clear, colorless and odorless drinking water, and one that does not possess these outward attractive physical properties. Among primitive peoples the question of water supply was never of pressing importance, except in arid and semi-arid regions. In these latter countries provision for securing and storing a supply of water was usually necessary. Consequently, springs were sought for, wells were dug, and cisterns constructed in order that a supply of water might at all times be available.

Wells were common in ancient Egypt, Greece, Assyria, Persia and India, and from the sanitary point of view they probably furnished a safer drinking water than could be obtained from the surface waters in the rivers and lakes. Wells of great antiquity may be found today in Egypt and India. Joseph's well at Cairo in Egypt is one of the most famous ancient wells, and was excavated in solid rock to a depth of 297 ft. The Chinese were familiar with the driving of artesian wells and pursued methods similar to those now in vogue in sinking them.

Where turbid surface waters had to be employed for drinking, domestic filters of unglazed earthenware or of sandstone were known to have been used by the ancient Egyptians and by the Japanese. Clarification of muddy water by the siphoning of the liquid from one vessel to another by the capillary action of porous material, such as a strip of cloth, and the consequent separation of the water from the suspended matter, was well known to the ancients.

As population became more dense and people began to congregate in cities, the need for larger volumes of water, than could be supplied by springs and wells, became urgent. Works for the collection, storage and conveyance of water were built for supplying many of the ancient cities, and the ruins of some of these works yet remain.

The ancient water tanks of Aden in Arabia for the collection of surface water from the gorges of the volcanic crater, at the bottom of which the old city was located, afford an example of early impounding reservoirs of an elementary type. These

tanks may have been built by Persian engineers as early as 600 B.C., or possibly by the Romans; but they without doubt antedate the Christian era.

The infiltration galleries for collecting ground water at Athens in Greece were probably constructed over 2,000 years ago, and the same method is still pursued to obtain a pure and satisfactory water supply. The rain-water cisterns of ancient Carthage (150 B.C.) were divided into several storage compartments, two of which were apparently used for settling or possibly filtering the water. Historians state that the city of Laodicea in Asia Minor obtained its supply from the River Caprus. The latter had its origin in springs. The water was conveyed to the city through a masonry aqueduct over 4 miles in length. A settling tank with double compartments formed a part of this water-works system. In Jerusalem underground cisterns were constructed, which were supplied through masonry conduits.

Probably no more elaborate system of public water supply was provided for any ancient city than that for Rome. Until 312 B.C., the Romans took their supply from the River Tiber and from springs and wells in the vicinity of the city. The increasing pollution of the river water, as well as the need for a greater volume of water, evidently induced the Romans to seek elsewhere for a supply. In consequence, three groups of springs in the volcanic plain on the left bank of the Tiber were made use of, and the water from them conveyed by aqueducts to the city. A fourth group of springs was also used which were located in the mountains of limestone formation somewhat more distant than the three other groups. This latter group furnished the best quality of water with which the city was supplied.

E. H. D'Avigdor, in his "Water Works of Ancient Rome" (*Engineering*, xxi, p. 403), interestingly describes the character of the Roman water supply as follows: "The Romans possessed three almost independent water services, for which they used water of different degrees of purity. The least clear and most loaded with sand, such as the Anio aqueduct supplied, was used for public baths and the watering of streets; the clearer water from Tepula and Alsietina served for tanks, fountains and washing troughs; while the very best (Virgo, Marcia and Claudia) was confined to drinking purposes, and these springs were undefiled even after the heaviest rains."

Rome was supplied with water from the above-mentioned

four groups of springs through nineteen aqueducts, which were built between 321 B.C. and 305 A.D. The aggregate length of these aqueducts was 381 miles.

There can be said to have been no real distribution system for the water entering Rome through the system of aqueducts. The aqueduct water flowed through small tanks in which the heaviest sand and gravel were deposited. To a limited extent the small distributing reservoirs (castella) near the city served a like purpose. From these reservoirs the water was distributed to cisterns, public fountains and private residences.

During the Middle Ages,[1] when disease played such havoc with the people of Europe, polluted water undoubtedly assisted in conveying infection. At the time of the fall of the Roman Empire many of the aqueducts built by the Romans at Rome and in the provinces were either destroyed or fell into disuse. The Moors in Spain during the ninth century constructed some important works, as well as repairing in the twelfth century some old Roman works.

Until 1183 A.D. Paris obtained its entire supply of water from the River Seine. As late as 1550, Paris used only 1 qt. of water per capita per day, and at the end of the seventeenth century was using but 2½ qt. per capita per day. With such small amounts of water being used one can easily imagine what sanitary conditions must have been.

London was first supplied in small quantities with spring water conducted through lead pipes, and masonry conduits. In 1582 a pump was erected on London Bridge to take water from the River Thames, and to deliver it through lead pipes to the city.

By the invention of the steam engine, pumping machinery of adequate capacity and power was made possible. The growth and development of water-works plants in reality dates from the eighteenth century. However, not until the latter half of the nineteenth century was very rapid progress made. The use of cast-iron pipe became general in about the year 1800, and gradually replaced the wooden mains formerly used.

Development of Modern Purification Plants.—The methods employed in securing and maintaining pure water supplies have been in a large measure governed by topographical and geological conditions. In different countries, where like physical conditions prevailed, similar lines of development have not always

[1] Turneaure and Russel: "Public Water Supplies."

been followed. Mr. Allen Hazen, in his book on "Filtration of Public Water Supplies," points out that "it is really marvellous how each country has met its problems of water supply from its own resources, and often without much regard to the methods which had been found most useful elsewhere. England has secured a whole series of magnificent supplies by impounding the waters of small streams in reservoirs holding enough water to last through dry periods, while on Continental Europe such supplies are hardly known. Germany has spent millions upon millions in purifying turbid and polluted river waters, while France and Austria have striven for mountain-spring waters and have built hundreds of miles of costly aqueducts to secure them. In the United States an abundant supply of some liquid has too often been the objective point, and the efforts have been most successful, the American works being entirely unrivalled in the volumes of their supplies. I do not wish to imply that quality has been entirely neglected in our country, for many cities and towns have seriously and successfully studied their problems, with the result that there are hundreds of water supplies in the United States which will compare favorably upon any basis with supplies in any part of the world; but on the other hand, it is equally true that there are hundreds of other cities, including some of the largest in the country, which supply their citizens with turbid and unhealthy waters which cannot be regarded as anything else than a national disgrace and a menace to our prosperity."

Since the above quotation was written, nearly two decades ago, many of the cities of the United States have done much to redeem the bad reputation which they had because of polluted and unwholesome water supplies. There is still much that can be done, however, toward improving present conditions, and as time goes on and population becomes more dense, the danger from polluted water increases.

Those impurities in water which render it turbid, and which for the most part are in suspension, make it unattractive as a drinking water. It was probably noted at an early period, that a muddy water, if allowed to remain quiescent for a short time, lost more or less of its suspended matter by settlement. In storing muddy waters for irrigation purposes this phenomenon could hardly have failed to have been noticed.

The appearance of a muddy water was obviously improved by

settlement, and its suitability as a drinking water enhanced. If clear spring waters could not be obtained for a public supply, resorting to surface waters was a necessity. Hence, we find evidence that in some of the ancient water-works systems provision was made for settlement of muddy waters in tanks or settling basins. The "castellæ" and piscinæ of the Roman aqueduct system evidently performed the function of settling tanks, whether originally intended for that purpose or not. In other ancient systems the evident purpose of tanks for settling purposes is more pronounced. Some of these works have been cited in the preceding pages.

These early devices for improving the quality of a water supply were crude and imperfect, and can only be recorded as the beginnings from which the really modern art of water purification sprang.

While sedimentation for the clarification of water may be looked upon as the earliest step in the art, nevertheless, it can only be regarded in most cases as preliminary to more complete methods. The straining action of sand and gravel was doubtless also noted by keen observers at an early date, in the same way that the clarification effected by settling tanks had been observed. The application of this principle to the public water supply of London was made in 1829. Filters of sand and gravel were also constructed for many of the Continental cities, more especially in Germany, where the principles underlying the action of filters of this type were carefully studied.

Mr. Allen Hazen has divided the history of water purification in the United States into three epochs, the first beginning with James P. Kirkwood's report on the "Filtration of River Waters" in 1866, resulting from his study of European practice; the second epoch commencing with the work of the Massachusetts State Board of Health at its Lawrence Experiment Station in 1887; and the third with the experiments on very turbid waters beginning at Louisville, Ky., in 1896, and continued at Pittsburgh and Cincinnati during the 3 or 4 years following. To these three epochs the author would add a fourth, which was introduced in 1908 by the experiments at Chicago and at Boonton, N. J., on the disinfection of water with hypochlorite of lime. The use of this latter compound has become widespread in the past 5 years throughout the United States, and its usefulness as a practical, efficient and economical agent in water purification,

under certain conditions, has been fully demonstrated. The sterilization of water with chlorine and its compounds, as well as the action of ozone and ultra-violet light, have been carefully studied during the past few years, especially in Europe. This phase of water purification is well established, and marks an important stage in the practical development of the art.

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CHAPTER II

CLASSIFICATION OF NATURAL WATERS

In order to understand intelligently the methods employed in water-purification processes, it is necessary to have a knowledge of the composition of the various classes of natural waters which require treatment. Although all the water which is now flowing on the surface of the ground, as well as that which is found in the ground, has a common origin, nevertheless, this water has acquired varied characteristics, which have been derived from the substances with which it has come into contact.

Rainfall.—From the water surfaces of the earth the sun is constantly evaporating enormous volumes of water, which are again condensed and precipitated upon the land and water surfaces as rain, snow or ice. That which falls upon the land surfaces of the earth is disposed of in different ways. Some of the water is evaporated again; some is absorbed by growing vegetation; some flows directly over the ground into the rivers and lakes; and some sinks into the soil to become the subterranean water of the earth. The amount of water precipitated on the surface of the earth in any particular region depends upon a number of factors. Warm air which is saturated with water in the form of vapor will only precipitate this moisture when sufficiently cooled. Air currents may be cooled by coming into contact with the cold surfaces of the earth, or by being forced upward by mountain ranges into colder air currents, or by mingling directly with colder air currents at lower levels. Precipitation of water will take place under these conditions.

The water thus precipitated is as near pure water as may be found under natural conditions. It probably will contain minute quantities of nitrogen compounds, such as ammonium salts, and some organic matter consisting of plant spores and bacteria. It will also contain in solution small quantities of the gases, oxygen and carbon dioxide.

As the descending rain or snow approaches the earth, it washes out of the atmosphere suspended particles of dust and thus purifies the air. These impurities will, of course, be found in the

rain water if collected before reaching the ground. Rain water caught in or near our large cities is much more impure than that collected in the open country. The impurities swept from the air are relatively in small amounts as compared with those acquired by the water after reaching the earth; but they are the first of the dissolved and suspended constituents to be taken up by the water, and are soon increased in amount and kind after the water reaches the ground.

SURFACE WATERS

Flowing Surface Waters.—The character of the land surfaces upon which rain or snow is precipitated naturally influences the proportion of the volume of water which finds its way over the top of the ground into the streams and lakes, and that which sinks into the ground to become a part of the underground water of the earth. If the land is mountainous, the rain rapidly runs off the steep slopes into the valleys, and quickly enters the rivers. If the soil is easily washed away, the water will carry with it large volumes of sediment, the composition of which will depend upon the geological characteristics of the denuded surfaces. On the other hand, if the precipitation occurs in a flat level country, a much larger proportion of the water will percolate into the soil. If vegetation is abundant, some of this water will be retained by the growing plants, and may sensibly retard the discharge of surface water into the natural water courses. In water which has been subjected to such conditions there will be found those dissolved salts with which the water has come into contact, as well as organic matter derived from vegetation.

Flowing surface waters may be characterized, therefore, as those which contain relatively small amounts of dissolved mineral compounds, more or less of suspended matter, depending upon topographical and geological conditions, and variable amounts of organic matter, likewise modified by the flora of the region.

Impounded Surface Waters.—Natural lakes and ponds are composed of water which has found its way directly through surface streams into the depressions in which these bodies of water lay, and of the water which has percolated for some distance through the ground and reappears as a surface water at lower surface levels. In a like manner the low-water flow of streams

is very largely maintained by the ground-water flow from higher levels.

In consequence, these lake waters are virtually mixtures of surface and ground waters, and their composition is, in a large measure, intermediate between them. Obviously local topography and the very important factor of sedimentation will materially affect their composition. They also have an important influence in rendering streams flowing from them more uniform in their discharge on account of the storage for water which they afford.

UNDERGROUND WATERS

The water which sinks into the ground does not all immediately pass through the subsoil to underground channels. Some is held by the surface soil, and in hot countries may soon be evaporated into the air; some of the water is absorbed by the growing plants; and the remainder sinks slowly or quickly through the ground depending upon the porosity of the latter.

The movement of this ground water is governed exclusively by the character of the strata with which it comes into contact. Impervious formations will block or divert the flow, while permeable strata will soon become filled and form the channel along which the underground stream moves. The velocity of flow is, of course, low, and depends upon the porosity of the soil in which the water is moving.

All of these factors have a bearing upon the dissolved constituents which will be found in an underground water. The suspended matter of the surface water will have been filtered out, whether in a relatively coarse form like sand and silt, or in a finer or colloidal condition like finely divided clay. On the other hand, the solution of mineral compounds containing sodium, potassium, calcium, magnesium, iron, aluminum, silica, etc., will have begun, and will continue as long as the water does not become supersaturated, and until "a chemical system of balanced values"[1] is attained for the various salts with which the water is in contact.

CONTAMINATION OF WATER

Natural Impurities.—For convenience the impurities in water may be divided into classes according to the sources from which

[1] Chase Palmer: "The Geochemical Interpretation of Water Analyses." U. S. Geological Survey *Bull.* 479.

they are derived. As has been previously noted water in the course of its flow over the surface or through the ground dissolves more or less material as well as carrying varying amounts in suspension. Provided no contact with the wastes of human life or activity has occurred, the acquired impurities might be designated as natural impurities, in distinction from those derived from sewage or manufacturing wastes. Water polluted from the latter sources is unfit for human consumption without purification as will be shown in a later chapter. Water which has not been subjected to such contamination may or may not be fit for drinking, depending entirely on the nature of the dissolved and suspended impurities which it may contain.

Dissolved Impurities.—The solvent action of water upon the seemingly insoluble constituents of the earth's crust is enormous, aside from its erosive action. The surface and subsoil, as well as the deeper rock strata, are all subjected to the decomposing action of the water with which they come into contact. Erosion assists the solution of the mineral compounds composing the various strata by reducing them to a finely divided state.

Disintegration of minerals like orthoclase, for example, is able to supply the bases sodium and potassium; dolomitic limestones may furnish calcium and magnesium; and clay ironstone and pyrite may give up iron, aluminum and silica. The acids with which the above bases are usually associated are carbonic, hydrochloric and sulphuric acids. These latter are derived from the disintegration of the minerals in the rock strata the same as are the bases. More or less carbonic acid is obtained directly from the air and from the oxidation of carbonaceous compounds in the breaking down of organic matter. Minute quantities of nitric acid may be derived directly from the air, and from oxidation of nitrogenous compounds. The oxidation of sulphur in minerals supplies sulphuric acid in many cases. Sulphates, chlorides, nitrates, carbonates and silicates exist as such in the earth's crust in enormous amounts, and whenever they come into contact with water are dissolved directly, and in amounts depending upon the supply available and the solubility of the salt being acted upon.

A classification of natural waters with respect to their dissolved constituents, which is based upon the chemical nature and the proportional amounts of the radicles present in solution, has been suggested by Mr. Chase Palmer in a paper entitled "The Geo-

chemical Interpretation of Water Analyses" (U. S. Geological Survey *Bull.* 479, 1911). He states that:

"Nearly all terrestrial waters have two general properties, salinity and alkalinity, on whose relative proportions their fundamental characters depend. Salinity is caused by salts that are not hydrolyzed; alkalinity is attributed to free alkaline bases produced by the hydrolytic action of water on solutions of bicarbonates and on solutions of salts of other weak acids."

According to his classification waters may possess five special properties as follows:

1. Primary salinity; that is salinity caused by the sulphates and chlorides of the alkalies sodium and potassium.

2. Secondary salinity; that is salinity produced by the sulphates and chlorides of the alkaline earths calcium and magnesium, or in other words permanent hardness.

3. Tertiary salinity; that is in reality an "acidity" arising from an excess of saline compounds over and above that due to primary and secondary salinity.

4. Primary alkalinity; that is an alkalinity produced by carbonates and bicarbonates of sodium and potassium, or in other words permanent alkalinity.

5. Secondary alkalinity; that is the property caused by alkaline earth bicarbonates such as those of calcium and magnesium, or in other words temporary hardness.

By various combinations of these properties five well-defined classes of waters are possible and are found in nature.

Class 1. Waters characterized by properties 1, 4 and 5 as stated above.

Class 2. Those showing properties numbered 1 and 5.

Class 3. Those exhibiting properties numbered 1, 2 and 5.

Class 4. Those marked by properties numbered 1 and 2.

Class 5. Those possessing properties numbered 1, 2 and 3.

"Surface waters appear to belong chiefly to the first three classes, class 4 is represented by sea water and brines, class 5 is exemplified by mine (acid) waters and waters of volcanic origin."

Suspended Impurities.—Intermediate between a solution, as it is commonly understood, and a suspension of finely divided particles, like sand for example, there may be so-called colloidal suspensions of certain substances in water which possess peculiar properties, and which are of considerable importance

in connection with water purification problems. For example, silica is found in this state in many natural waters, especially those showing primary alkalinity, *i.e.*, waters containing sodium and potassium carbonates. The silica compounds characteristic of the clays show a marked tendency toward the colloidal state, and render the problem of the purification of turbid waters of this class almost a problem in itself.

The enormous amount of the heavier sediment carried by many rivers consists largely of sand and clay. Any reduction in the velocity of flow of the water laden with such sediment causes it to be deposited on the bed of the stream. Gravel and sand bars are thus formed in river beds, which may be shifted from one point to another by any sudden increase in the velocity of the current, such as might be produced by a flood. Lighter material like clay is more slowly deposited and more quickly moved again by any change in the rate of flow of the water. It thus happens that almost all surface waters carry varying amounts of suspended material in them which depend upon the velocity of the currents of water and on the character of the bottom and shores. The action of the wind on large bodies of water like the Great Lakes, for example, may stir up the sediment on the bottom and render the water, near the shore especially, quite turbid.

Compounds of iron and of manganese are not infrequently met with in the colloidal state in natural waters, and offer some of the most interesting phases of water-purification work. Organic matter found in natural waters is probably always present in this form to a greater or less degree. Vegetable stain produced by humic substances is a marked characteristic of a very large class of natural surface waters, and as such has received considerable attention in the study of purification problems.

How the removal of these impurities, whether in solution or in suspension, is effected will be discussed under the description of the various methods of purification now employed, rather than in this place. It is only desired to emphasize the varied classes of impurities which may be found in natural waters, and which are virtually "natural impurities," as distinguished from those derived from sewage and manufacturing wastes. This waste material may furnish similar classes of polluting compounds to those derived naturally, but their origin usually justifies their consideration separately.

Whether the dissolved salts usually found in natural waters are objectionable depends upon the use to which the water is to be put. A certain amount of the chlorides, sulphates and carbonates of sodium, potassium, calcium and magnesium are by no means deleterious, and may possibly be beneficial in a drinking water. On the other hand, if too great amounts are present these dissolved salts render the water unfit for domestic and industrial uses, and actually cause financial losses of no small amount to those obliged to use them. Compounds of iron or of manganese are especially objectionable, and not infrequently have caused waters containing them to be abandoned as sources of supply. Excessive amounts of the fixed alkalies either as salts of the strong acids like hydrochloric and sulphuric, or of the weak acids like carbonic, make a water unsuitable as a public supply.

Microscopic Plant and Animal Life.—In many of our natural waters a luxuriant growth of algæ and diatoms is found at certain seasons of the year. The character of the mineral and organic constituents of the water, as well as the conditions of light and temperature materially affect the extent of these growths. They occur in both still and running water. Accompanying the growth and also the decay of certain of these organisms bad odors and tastes are not infrequently developed, and where this occurs in public water supplies, they become a nuisance entirely out of proportion to their number and size. Certain microscopic animal forms may also produce troubles of this same character.

The odor of growth appears to be due to secretions of an oil-like character which they produce, and is usually somewhat characteristic of the special organism producing it. When the organisms are in large enough numbers they are capable of giving an odor to large volumes of water, and not infrequently spoil the taste and odor of the whole of a public water supply. Odors of decomposition are usually very offensive, and notably so in the case of the "blue green algæ" or Cyanophyceæ.

It is not probable that impurities of this nature in natural waters produce disease in human beings, when swallowed in drinking water. They are very objectionable if they produce a marked odor, and in such cases are most frequently complained of in public water supplies.

Bacteria.—Even lower in the scale of plant life than the diatoms and the algæ are found the bacteria. They are present

in all natural waters, being the more numerous in surface waters, and much less so in ground waters. By far the larger number of the various species of bacteria play a beneficent rôle in the economy of nature, and appear absolutely essential to many of the normal processes of development of both plants and animals. A few species, however, are associated with disease in animals and in human beings. So far as the contamination of water is concerned, it is only the organisms capable of producing pathologic conditions in man that are of interest. These virulent forms usually reach our natural waters through the medium of domestic sewage and manufacturing wastes. This class of impurities is considered in more detail in the next section.

IMPURITIES DERIVED FROM WASTE MATERIAL

Sewage.—The water carriage of waste material of human and animal origin has become, in those countries which pay any attention to problems of sanitation, the most common method for its transfer to some point of ultimate disposal. Wherever public water supplies are installed, a system of sewers will of necessity follow. Hence the disposal of large volumes of fouled water has become a problem of great difficulty, and one that yet awaits a completely satisfactory solution.

It is obvious that some method, even though it is not entirely satisfactory, must be used to get rid of this polluted water or sewage, and the easiest way has been to turn it into the natural water courses. In this manner much of the surface water on thickly settled land areas has become polluted with material dangerous to the health of human beings, who unwittingly or of necessity drink the water thus contaminated. Since disease has been found to originate so largely from specific plant and animal forms, microscopic in size, which, having produced the disease, are discharged from the body chiefly in the excreta and the urine, the conveyance of disease through sewage to water has been pretty definitely proven.

Ground waters as well as surface waters may become polluted by sewage. The discharge of sewage on the surface of the ground or into cesspools, or the leakage or overflow of vaults, may furnish the dangerous pollution to well and spring waters by direct percolation through fissures in the rock strata, or by more indirect routes through the soil itself.

Manufacturing Wastes.—In many industries there remains after the manufactured product has been completed, a great deal of waste material, which for economic reasons it is not worth while to work over. Much of this material is in suspension and solution in relatively large volumes of water. Its disposal by the easiest method is to dump it into the nearest body of water. Water fouled with such material is totally unfit for human consumption. Frequently the material renders even the best methods for the purification of domestic sewage inadequate, and its proper disposal becomes a special problem in almost every case.

The waste liquids from textile works, dye works, straw board factories, paper mills, abattoirs, meat packing establishments, dairies, etc., furnish material which is obviously difficult to dispose of, and which must pollute in the foulest manner any natural water into which they may be turned.

NATURAL METHODS OF PURIFICATION

Sedimentation.—Some of the impurities which a natural water acquires in the course of its flow, may be lost under certain favorable conditions. For example, a water laden with suspended clay or fine sand will deposit this material as soon as the velocity of the water is sufficiently retarded as was previously explained. This process of sedimentation is one of the most important of the natural methods of purification, and plays an important part in our artificial methods as well. In flowing streams the deposition of sediment is intermittent, being active during low stages of the stream when the rate of flow is relatively low, and much diminished or practically nil in flood periods. At such times the scouring action of the current causes much that has been deposited to be again placed in suspension, and thus carried further toward its ultimate disposal in the sea. In this way the immense deltas at the mouths of rivers like the Mississippi and the Nile are formed.

Effect of Sunlight.—The purifying action of sunlight on certain vegetable compounds in colloidal suspension, such as the brown coloring matter in many of the streams and lakes in the north central and northeastern parts of the United States, is worth mentioning in this connection. A certain amount of bleaching out of this coloring matter is apparently effected when this class

of waters are impounded in natural lakes or artificial reservoirs. Oxidation of the carbonaceous matter probably occurs, and sedimentation in the quiet water undoubtedly assists in the clarification.

Precipitation of Compounds from Solution.—Dissolved salts are not usually readily removed once they have gone into solution. The chlorides, sulphates and nitrates of either sodium, potassium, ammonium, calcium or magnesium will be retained on account of their great solubility. Carbonates and bicarbonates of the fixed alkalies, as well as of ammonia, are also very soluble. On the other hand, the bicarbonates of the alkaline earths have a rather limited solubility and may be deposited from solution if the excess of carbon dioxide, which is necessary for their retention in solution, is in anyway removed. Ground waters in particular may become heavily charged with bicarbonates and, on being brought to the surface where the pressure is diminished, will lose some of their free carbon dioxide and deposit their monocarbonates, which are much less soluble. This is particularly true of calcium carbonate. Magnesium carbonate, however, is considerably more soluble.

Oxidizable salts like ferrous sulphate, ferrous carbonate, and corresponding salts of manganese occurring in ground waters may be deposited from solution on exposure to the air. Such purification can be hastened by aeration and thus render some unsuitable deep well waters entirely acceptable as a source of water supply.

"Acid mine waters" are usually contaminated with dissolved iron compounds, which not infrequently find their way into surface streams. As much of this iron may be in an unoxidized state, the exposure to the oxygen of the air, and to that dissolved in the surface water, soon converts the iron to the form of the insoluble ferric oxide. Organic matter in colloidal suspension may retard the precipitation of the iron, and cause the latter to assume a colloidal state itself.

In waters in which the alkalinity is due to sodium and potassium carbonates, colloidal solutions of silica and alumina are sometimes found, and on account of their slight solubility may be deposited, should this "primary alkalinity" be diminished. Such a diminution can be effected if waters of this class come into contact with chlorides and sulphates of lime and magnesia. These latter salts will react with the fixed alkaline carbonates,

forming carbonates of lime and magnesia and the chlorides and sulphates of sodium and potassium. The latter salts are without power to assist in holding the silica in solution.

Filtration.—The natural filtration of water through the soil effects a high degree of purification provided the ground is of the right character. Sand and gravel, when not too coarse, afford an excellent purifying medium. Suspended impurities, organic matter and oxidizable salts are removed as a result of the straining action, and the chemical and biological changes induced during filtration. The action of both sedimentation and filtration in purifying natural waters is perfectly normal, and one which is constantly going on. To these agencies we owe the potability of most of our ground waters, and by a study of the principles underlying these natural processes we have been able to design and operate our modern water-purification plants.

PURIFICATION BY MEANS OF MINUTE PLANT AND ANIMAL ORGANISMS

Thus far in considering natural methods of purification only those agencies have been especially noted which are effective without the intervention of organized plant and animal life. In the cycle through which inert mineral matter passes into organized matter, and then back again into inorganic compounds, life in some of its most marvellous forms plays an important and essential part. These forms belong both to the animal and vegetable kingdom, and for the most part are microscopic in size. The borderland between plant and animal, in these almost invisible organisms, is extremely ill-defined; but no matter how classified their importance in the economy of nature is fundamental.

All living organisms which float about in water between the surface and the bottom are designated by biologists as "plankton." They are moved about by the currents and the wind chiefly, although they have slight powers of locomotion. They also possess the peculiar ability to remain suspended in the water with little effort on their part. The plankton can be divided into two general classes:

"the food producers or plants, which assimilate inorganic matter and build up organic compounds by means of their chromophyll coloring matter; and the food consumers or animals, such as the microscopic protozoa, rotifera, etc., together with the larger ones up to the fishes."

In the lecture from which the above quotation was cited, Dr. Marsson[1] concisely epitomizes the relations of these two groups by stating that the

"vegetable component of the plankton is the fundamental food supply or condition of existence for all aquatic life. It comes from the products of the decomposition of the albumen which finds its way into the water from decaying animals and plants, as well as from sewage. The self-purifying power of natural waters is merely the maintenance of the proper equilibrium between retrogressive and progressive metamorphosis."

The groups of microscopic plant forms known as the algæ, diatoms, fungi and bacteria exist in enormous numbers in all natural surface waters, and to some extent in ground waters. The algæ and diatoms through their peculiar cellular structure are living laboratories in which light is the energy which tears the carbon from carbonic acid, and the nitrogen from its simpler compounds and converts them into starch, sugar and albumen. Thus oxygen is liberated and becomes available for oxidizing organic matter and preventing putrefactive changes.

The fungi and bacteria find their nutriment in dead organic matter, and are the primary agents for its decomposition into simpler compounds. Bacterial activity is associated with the using up of large amounts of oxygen, where the latter is available; and in such cases non-putrefactive disposal of contaminating impurities in water is in process in distinction to putrefactive changes where the oxygen is not present.

The bacteria are the natural food for many of the microscopic animal organisms. The latter include the protozoa, infusoria and metazoa, and where these organisms are found in abundance, bacteria and food for bacteria will also be present. They thus become indexes of pollution in water, quite as indicative as the bacteria themselves. When the food supply is gone they must die also.

Since the smaller plant and animal organisms are the source of food for the fishes, and they in turn for human beings, the cycle of matter from man through human wastes to mineralized

[1] Max Marsson: "The Significance of Flora and Fauna in Maintaining the Purity of Natural Waters, and How They are Affected by Domestic Sewage and Industrial Wastes." *Eng. News*, Aug. 31, 1911. Trans. by Emil Kuichling.

compounds and back again to man is complete. The natural methods of self-purification of water are going on ceaselessly and effectively, but the agencies ordained for this purpose must have time and opportunity to do their work.

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CHAPTER III

TRANSMISSION OF DISEASE THROUGH DRINKING WATER

The discharge of sewage and waste material of all kinds into the streams and lakes obviously affords ample opportunity for disease-producing organisms to enter the sources of most of our public water supplies. Those diseases peculiar to the intestinal tract of the human body are the ones most likely to be disseminated in consequence of this common practice; and hence, typhoid fever, cholera, dysentery and gastro-intestinal disturbances have come to be regarded as derived in a large measure from polluted drinking water, wherever these diseases are endemic or even epidemic. Anyone or all of these diseases may be transmitted in other ways, but where they are widespread, some common carrier of infection is generally found to be the source, and a common drinking-water supply usually offers the most favorable opportunity, for transmitting the disease.

Probably most of the diseases transmitted by water are of bacterial origin. The "spirillum choleræ" of Asiatic cholera, the "bacillus typhosus" of typhoid fever, and the "bacillus dysenteriæ" of dysentery have all been found in contaminated drinking water. Pathogenic protozoa may also produce certain diseases, and in the case of one form of dysentery, an amœba is known to be the cause. If the theory advanced by Sedgwick and MacNutt, that inflammatory diseases of the respiratory organs may be also to some extent water-borne, is accepted, then polluted water supplies are chargeable with another group of diseases particularly prevalent among all classes of people.

The virility of disease-producing organisms upon their entrance to a water is of importance with respect to the real danger which they possess. This ability to live and retain their vitality in a medium foreign to their natural habitat is also of consequence, for if the power to reproduce the disease is soon diminished and eventually destroyed, then the length of time before their vitality is lost is of the utmost importance. Much experimental work has been done to determine the period elapsing before certain pathogenic organisms die, when placed in water under varying

conditions. Laboratory experiments throw some light on this problem, but are not usually conclusive, because of the artificial conditions imposed. In almost all the experimental work the number of organisms diminish in time, and usually very rapidly. This may be the result of a decreasing food supply, or to toxic compounds eliminated in the course of growth or decay, which kill off rapidly the less resistant organisms.

The presence of pathogenic forms in a drinking-water supply denotes, of course, all the possibilities of dangerous infection. Nevertheless, sanitarians have of late regarded the number of such organisms, and the length of time which they may have been in the supply, as factors of much importance in the epidemiology of disease. This quantitative feature is of considerable significance in connection with the quality of a water obtained by the methods commonly employed in the purification of polluted waters.

Spirillum Choleræ and Bacillus Typhosus.—The cholera spirillum and the typhoid bacillus are the pathogenic organisms which have been most studied in water-borne diseases. It is very doubtful whether either of these organisms will multiply outside the body, or in impure water. The cholera spirillum is not very resistant to adverse conditions outside the human body. It is killed in 10 min. by a temperature of 60°C., easily destroyed by chemical disinfectants, and does not long retain its vitality in association with the ordinary saprophytic bacteria in the water.

The typhoid bacillus is probably more resistant than the cholera organism to outside influences. Laboratory experiments have demonstrated that the typhoid bacillus will live in sterile water in glass vessels for 3 months, and in unsterilized ground and surface waters for several weeks. Jordan[1] showed by his experiments with typhoid cultures placed in sacks of collodion and parchment and suspended in flowing water, that they would retain their vitality under natural conditions for at least 4 or 5 days. Mr. Geo. A. Johnson's experiments at Columbus, Ohio,[2] in which he modified Prof. Jordan's technique, showed that the ability of the bacteria to pass through the walls of the parchment sacks, might indicate that conclusions drawn from the disappearance of the bacteria in Jordan's experiments, were

[1] Jordan, Russel and Zeit: *Jour. Infect. Diseases*, 1904, 1, p. 641.

[2] *Eng. Record*, vol. 52, Sept. 23, 1905.

somewhat misleading. If the organisms actually escaped from the sacks, failure of samples withdrawn from the latter to develop typical cultures, did not necessarily mean that the typhoid bacilli had died. Jordan concludes,[1] that:

"It is possible that water may continue to be the vehicle of infection during a much longer period (than 4 or 5 days), but the available data point to a comparatively short duration of life of the specific germ in the water of flowing streams."

Houston[2] has shown that samples of Thames River water inoculated with a typhoid emulsion and stored at temperatures ranging from 32°F. to 98.6°F., developed negative tests for typhoid in 9 weeks at the low temperature, and in 2 weeks at the highest temperature. Intermediate temperatures gave negative results in conformity with the results stated above, viz., the higher the temperature of the water, the shorter the period of life of the organism. The history of typhoid epidemics tends to confirm in a measure the data obtained in these experiments. It emphasizes the protective value of ample periods of sedimentation and storage of polluted waters used as public supplies.

Isolation of Cholera and Typhoid Organisms from Water.—The actual isolation of these two organisms from polluted water has been accomplished only in a comparatively few well-authenticated cases. In the case of cholera the organism is discharged from the intestines in enormous numbers, but not in the urine. Its appearance in sewage and polluted water would, therefore, be expected, and has been demonstrated. In 1892 Dunbar isolated the spirillum of cholera from the polluted water of the Elbe, during the epidemic in Hamburg. Koch[3] also reports its isolation from the water of two Altona reservoirs supplied also from the Elbe.

The isolation of the typhoid bacillus from natural waters also offers a great deal of difficulty, although it probably is more virile and capable of living longer in natural water than the cholera organism. The bacilli of typhoid fever are discharged from the human body both in the urine and the feces. From 9 to 14 days after infection has taken place are required before the disease fully develops. This characteristic feature of typhoid fever makes the tracing of infection through natural waters much

[1] E. O. Jordan: "General Bacteriology."
[2] *Eng. Record*, vol. 65, June 1, 1912, p. 608.
[3] *Zeit. für hygiene und Infect. Krank.*, 14.

more difficult, for although the bacilli may have been present and caused the disease, they will have probably disappeared before suspicion is thoroughly aroused as to their possible presence in the water. Comparatively few cases have been recorded, therefore, in which the bacillus has been isolated and shown to have been the probable cause of a case of typhoid fever.

The work of Mr. D. D. Jackson and his associates in improving methods[1] of technique for the differentiation of the colon-typhoid group of bacilli offers considerable hope that the isolation of the typhoid fever bacillus may yet be accomplished with more certainty and ease. Mr. Jackson states that he has isolated B. typhosus from a river water used as a source of water supply, from a local private water supply and from two points in the Hudson River.

Other Water-borne Diseases.—Intestinal diseases and some gastric troubles may be and probably frequently are caused by organisms found in water. Among infants this perhaps is truer than with adults. Epidemics of diarrhea and dysentery are not uncommon and have been traced to impure drinking water. The possible infection of a water supply by anthrax (B. anthracis) derived from animals sick with the disease produced by this organism is rather remote, but not impossible. It is of more theoretical interest than practical that the pathogenic organisms B. anthracis and B. tetani[2] have both been isolated from river water by Zeit and Fütterer; but it goes to show the possibilities of water-borne infection. Sewage-polluted waters may contain all known pathogens as well as saprophytes, and what rôle the latter forms may play in disease is by no means a settled question.

The relation between pneumonia, bronchitis and other inflammatory diseases affecting the respiratory organs and polluted drinking water has been noted above. The ascertainable facts relating to this phase of water-borne diseases are few and difficult to satisfactorily classify. The data already collected by Dr. W. T. Sedgwick and his associates are extremely valuable, and further confirmation of their deductions is hoped for.

[1] D. D. Jackson and T. W. Melia: "Differential Methods for Detecting the Typhoid Bacillus in Infected Water and Milk." *Jour. Infect. Diseases*, vol. 6, No. 2, April 1, 1909.

[2] "Report of the Sanitary Investigation of the Illinois River and Its Tributaries." Ill. State Board Health, 1900, p. 85.

EPIDEMICS OF WATER-BORNE DISEASES

Cholera

Probably no disease is more truly characterized as a "filth disease" than is cholera. In certain parts of India it may be said to be endemic. Explosive outbreaks are not uncommon, and the spreading of the disease by contact is probably constantly going on. The insanitary nature of the personal habits of the lower classes of natives affords ample opportunity for transmitting infection, and the streams and lakes frequently serve as carriers.

In 1817 a violent epidemic of cholera broke out in Jessore in Bengal, which rapidly spread over a larger part of British India. It continued unabated for 3 years, and then began to spread into China and Persia. In 1823 the disease had reached Asia Minor and Russia. For the next 7 years it did not advance westward any further, but a fresh outbreak in 1830 in Russia caused the disease to spread all over the latter country and into northern Europe and the British Isles. During the next 5 years it spread southward, invading northern Africa.

Another epidemic started in India and China in 1841, reaching Europe in 1847; another began in 1850 and entered Europe in 1853, and was carried across the Atlantic to North and South America, where it was particularly severe. The epidemic of 1865–66 was less extensive than its predecessors. Since 1832 eight epidemics of cholera have occurred in the United States, the last being in 1873.

With a better idea of the true cause for this disease in particular, and with improved methods for combating infection and contagious diseases in general, cholera has not been widely prevalent in Europe or the United States for a great many years. Constant vigilance is required for its suppression, however, and only by prompt action, where sporadic cases are discovered, have the health authorities prevented epidemics. How many of these epidemics have been directly transmitted through drinking water, it is impossible to know; but that water acted as a carrier to a greater or less extent in many of them is extremely probable.

In the period from 1831 to 1873, 373,000 people died in Prussia of Asiatic cholera, and in 1886 alone 114,000. In 1892, 1,634 persons died from this disease in Prussia, and from the Hamburg

epidemic in this same year 8,616 deaths resulted. In 1910 Germany had but 10 cases of cholera.[1]

Circumstantial evidence of a very convincing character has been collected, which proves that some cholera epidemics were water-borne, and probably no discussion of this subject is complete without mentioning the disastrous Hamburg epidemic which occurred in 1892–93. This city was using unfiltered water from the River Elbe, which was contaminated by the sewage of over 800,000 people. During the fall of 1892, 17,000 cases developed, resulting in 8,600 deaths. In fact, wherever the drinking-water supply was either filtered or obtained from some source other than the river, few or no cases resulted.

In 1887 the city of Messina, Sicily, suffered from an epidemic of cholera, during which 5,000 cases and 2,200 deaths resulted. An investigation showed that water purposely diverted from a conduit, conveying water to the city, ran into pools, which were used for washing soiled clothing by the Messina washerwomen. Much of this water found its way back into the open conduit, and passed into the city. It was also found that the unglazed tile used to distribute the water in the city were broken, and that leakage from joints was common. Sewers laid on top and parallel with the water mains were in a like condition and offered excellent opportunity for further contamination. After a supply of pure water, carried in tank ships from the mainland, was provided for drinking water, the epidemic ceased at once.[2]

A similar outbreak of cholera in 1884 in Cuneo, Italy, which resulted in 3,344 cases, was traced to a like cause, viz., washing infected linen in a brook emptying into a public water supply.[2]

Typhoid Fever

The prevalence of typhoid fever in civilized countries, where no little attention is paid to matters of sanitation, seems at first thought surprising. But not until 1880 was the organism which causes this disease discovered by Eberth in the spleen of persons dying from typhoid fever. Since it seems doubtful that this disease, as it develops in human beings, can be reproduced in animals, the evidence that the Eberth bacillus is the true cause

[1] Dr. Arthur Lederer: "The Modern Sewage and Water Problem." *Clinique*, August, 1912.

[2] W. P. Mason: "Water Supply."

for the disease has been only slowly accumulating. Another factor only recently discovered is that persons showing no clinical symptoms of the disease are genuine "culture factories" for producing the bacillus and for its dissemination.

Epidemics resulting from these "typhoid carriers" have been satisfactorily traced. The existence of such persons explains in some measure the continuance of the disease, and its persistence. The transmission by direct contact, by flies, by milk and by water has been proven in scores of cases.

While there may be some extenuating circumstances for the continued presence of typhoid fever to the extent to which it still exists in the United States, nevertheless it constitutes a national disgrace, of which our sanitary authorities and people as a whole should be heartily ashamed. Typhoid fever has been practically stamped out in Europe as the following table will show.[1]

Unit of comparison	Aggregate population	Deaths per 100,000 from typhoid fever, 1910
Thirty-three principal European cities in Russia. Sweden, Norway, Austria-Hungary, Germany, Denmark, France, Belgium, Holland, England, Scotland and Ireland..	31,590,000	6.5
Fifty American cities of 100,000 inhabitants or over	20,250,000	25.0
Excess of deaths, typhoid fever in American cities per 100.000 population	....	18.5

In three-fifths of the population of the United States included in the registration area for mortality statistics as compiled by the Census Bureau, there occurred 12,673 deaths from typhoid fever in 1910, or a death rate of 23.5 per 100,000 of population. Assuming the same proportion of deaths in the unregistered sections as in the registered, then there were 21,120 deaths from this disease, representing probably 200,000 cases.

The following diagram (Fig. 1) illustrates the prevalence of this disease in the United States as compared with certain countries in Europe.

[1] ALLAN J. McLAUGHLIN: "Sewage Pollution of Interstate and International Waters." Public Health and Marine Hospital Service, Hygienic Lab. *Bull.* 83, March, 1912.

Dr. J. F. Anderson[1] concludes from his study of typhoid fever epidemics due to contaminated water, that they are characterized by:

(*a*) A general distribution of cases throughout the area supplied by a particular water.

(*b*) By the explosive onset of the outbreaks.

(*c*) By the trouble occurring in the late winter or spring.

(*d*) By the comparative freedom from the disease of persons not using the suspected water.

(*e*) By evidences of sources of infection found by an inspection of the watershed.

(*f*) By the outbreak beginning or ending after a change in the water supply.

(*g*) And by indications of the pollution of the water when analyzed.

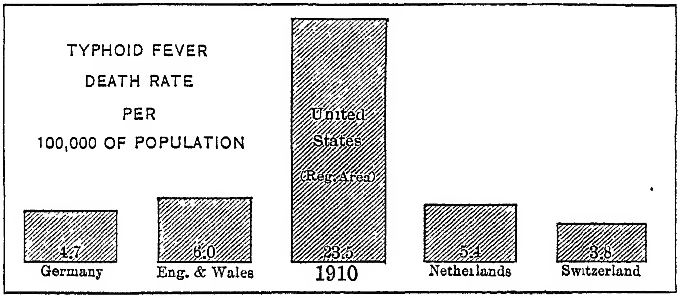

Fig. 1.—Typhoid fever death rate in various countries.

As an illustration of one or more of the above features of typhoid fever epidemics, the one which came under the author's personal observation, may be cited as quite characteristic. A town located on the Ohio River was supplied, together with two other communities, with water from a tributary of the Ohio River. This tributary flows into the Ohio River a short distance above the town. The water supply was treated with sulphate of iron and lime, and settled in a series of basins, from which the water flowed by gravity to the service pipes of the three communities. On account of a fire the pumping station was partially disabled, and the town located on the Ohio River was cut off from this supply for a period of about 5 weeks; but the two other communities continued to receive water from this same

[1] "A Symposium on Typhoid Fever." *Amer. Jour. Public Hygiene*, May, 1909.

source. In order to obtain a supply of water for this town, a pumping station which had been out of service for some time, and which drew its supply of water from the Ohio River, was started, and pumped Ohio River water into the service pipes for 35 days, covering parts of the months of October and November. Just above the intake of this pumping station is a small creek, which drains a ravine. Along this ravine and just above the pumping station were located quite a number of houses, about which the sanitary conditions were bad. While this pumping station was in service, a small artificial pond, located near the head of this ravine and connected with a summer amusement resort, was emptied. This water was discharged into the Ohio River just above the pumping station intake.

An epidemic of typhoid fever began in the town supplied with Ohio River water about the middle of November. The author was not called in until about one month later, and at that time 135 cases had been reported.

An examination of the milk supply showed insanitary conditions about many of the dairies, but cases of typhoid were not found to be confined to any particular milk route, but were general all over the town. The milk supply was evidently not the source of the infection. The town had many wells, but the distribution of the cases was too uniform to attribute the infection to any particular locality in the town.

At the time the author was making the investigation, the Ohio River supply of water had been stopped, and a return to the supply formerly used had been made. It was not, therefore, possible to obtain much chemical or bacteriological evidence of the character of the water which had been taken from the Ohio River. It was ascertained, however, that no epidemics of typhoid had occurred in the two other communities which had continued to receive their regular supply of partially purified water. Neither was there known to have been any more than the usual number of cases in a town on the opposite side of the Ohio River, and taking its supply from the latter stream.

The logical cause for the epidemic seemed to be, therefore, the temporary pumping of a polluted water from the Ohio River, which rapidly infected many of the persons who drank it. Moreover, on account of the explosive character of the outbreak, the pollution of the water was probably quite direct. The germs may have been washed into the Ohio River, and from thence

passed into the intake of the pumping station at the time the pond in the summer resort was emptied, or following a flushing out of the creek by rains. The insanitary conditions along the creek, and the negative evidence obtained in investigating the milk supply, all led to the conclusion that it was a water-borne epidemic of typhoid fever that had occurred.

A somewhat similar water-borne infection, which caused an epidemic of typhoid fever in Columbus, Ohio, in 1903–04, resulting in 1,606 cases and 162 deaths in 3 months, probably originated from the pollution of the public water supply drawn from the Scioto River.[1] Cases of typhoid fever at the State Hospital were known to have existed 10 days prior to the outbreak in Columbus. The sewage from this institution entered the Scioto River not far from the Columbus water-works intake. The suddenness of the outbreak is shown by the following table:

TYPHOID FEVER EPIDEMIC, COLUMBUS, OHIO

	Cases	Deaths	Rate per 100,000 of population
December, 1903	40	4	34
January, 1904	725	35	300
February, 1904	798	94	805
March, 1904	83	33	283

Some of the more important typhoid fever epidemics which have been traced to infected water are listed below:

Place	Year	Population	Number of	
			Cases	Deaths
Caterham, England	1879	5,000	352	21
Plymouth, Pa.	1885	8,000	1,104	114
Tees River Valley, England	1890–91	251,976	1,330	100
Lowell, Mass	1890–91	77,696	2,855	217
Lawrence, Mass.	1890–91	44,654	1,792	137
Worthing, England	1893	16,000	1,411	168 (Wells)
Grand Forks, N. D.	1893–94	6,000	1,245	96
Maidstone, England	1897	33,830	1,928	150 (Springs)
Ithaca, N. Y.	1903	18,000	1,350	82
Butler, Pa.	1903	13,000	1,348	111

[1] *Jour. Mass. Assoc. Boards of Health*, vol. 14, May, 1904.

Comparatively recent outbreaks of water-borne typhoid fever, or a gradually increasing prevalence of this disease, which forced the authorities to provide remedial measures, have occurred at Erie, Pa., Niagara Falls, N. Y., Evanston, Ill., Coatesville, Pa., Ironton, Ohio, Winnipeg, Canada, Rockford, Ill., Memphis, Tenn., Council Bluffs, Iowa, and Omaha, Neb.

The pollution of the Great Lakes in the United States by the cities built upon their shores is to a large extent local; but since these communities draw their water supply from the same source, the problem of preventing the drinking water from becoming contaminated, and still obtain a satisfactory disposal of the sewage, is a troublesome one. The common remedy of extending the water supply intakes out from 3 to 5 miles from the shore has been resorted to with fair success, and with marked decreases in the typhoid death rate in most cases. The City of Chicago has diverted a large part of its sewage through a drainage canal into the Illinois River, thus keeping a constantly increasing volume of sewage from polluting the lake water farther and farther from the shore line. Other lake cities are contemplating partial purification of their sewage in order to conserve the purity of their water supplies.

The effect of the wind on these large bodies of water in causing currents, the influence which the shore lines may have on these currents, and the movement of ice polluted with sewage from the shore out into the lake in the spring months are all factors which may at times be the cause for water-borne epidemics.[1]

Even ground-water supplies, which through carelessness or ignorance are not properly protected from pollution after being drawn to the surface, are not infrequently the distributors of infectious material. A rather remarkable case was recently described as occurring at Lincoln, Neb.,[2] in which a water from a well 60 ft. in depth became contaminated by leakage of sewage from a broken sewer. The escaping sewage found its way into the ground and into an abandoned pipe which had been forgotten and which connected directly with the well.

An outbreak of typhoid fever occurred during September, October and November of 1911, from which six deaths resulted.

[1] D. D. Jackson: "Chlorination at Cleveland, O." *Eng. Record*, vol. 65, June 15, 1912.

[2] "A Polluted Well at Lincoln, Neb." *Eng. Record*, vol. 65, June 15, 1912, p. 614.

About the middle of December a severe outbreak of bowel trouble, during which there were several thousand cases, took place. This was followed about Dec. 20 by a second epidemic of typhoid fever, during which 300 cases were reported.

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CHAPTER IV

THE EFFECT OF IMPROVED WATER SUPPLIES UPON HEALTH

As the relation between impure water and disease becomes better understood and appreciated, more attention is being given to the quality of public water supplies. Sometimes the method pursued is to seek a purer water from some uncontaminated source, or to render a polluted supply better by some process of purification. By substituting a pure drinking water supply for an impure one, the reduction in the death rate from water-borne diseases has been notable, and the improved health of the community has usually been demonstrated beyond question.

In the United States the typhoid fever death rates undoubtedly furnish the best indicators of the quality of public water supplies. The following table taken from Dr. Geo. M. Kober's paper on the "Conservation of Life and Health by Improved Water Supply" summarizes the statistics of 61 cities in the United States for the years 1902–06.[1]

Mean Typhoid Fever Death Rate from 1902–06 per 100,000 of Population for Cities Using Various Classes of Water

4 cities using ground water from large wells	18.1
18 cities using impounded water and conserved rivers or streams	18.5
8 cities using water from small lakes	19.3
7 cities using water from the Great Lakes	32.8
5 cities using both surface and underground water	45.7
19 cities using polluted river water	61.1

From the same paper is reproduced a diagram (Fig. 2) showing in more detail the typhoid fever death rates in different cities according to the character and the source of their water supply.

Spring waters, ground waters from wells, and filtered waters evidently furnish the safest supplies. Surface waters, whether from streams or lakes, may furnish safe drinking water, but they are much more likely to be polluted.

Probably the most striking effects in reducing typhoid fever have come from the purification of polluted supplies. By filter-

[1] *Eng. Record*, vol. 57, June, 1908.

ing an impure water supply marked reductions in water-borne diseases have almost invariably resulted. Even where the

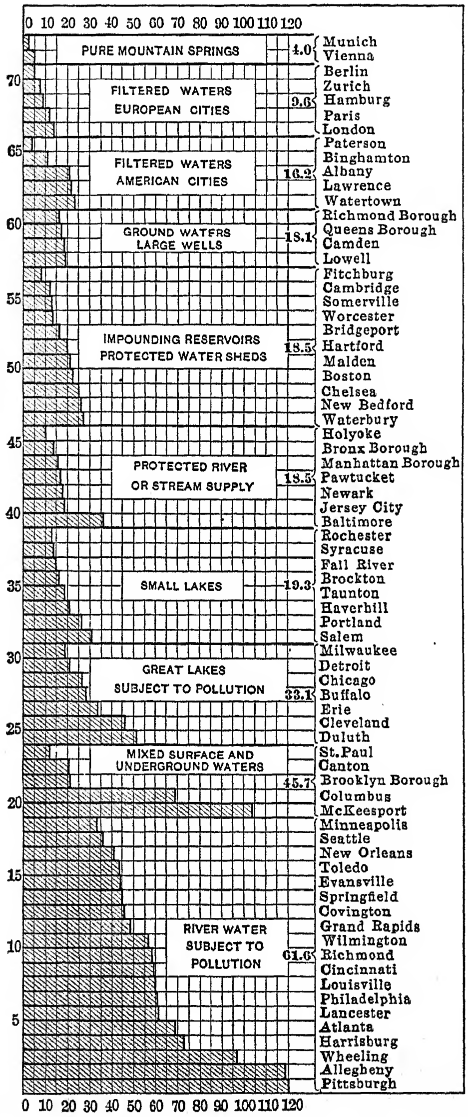

FIG. 2.—Typhoid fever death rate according to water supply.

community has been only partially supplied with purified water, the effect on the typhoid death rate has been noticeable.

TYPHOID DEATH RATES PER 100,000 OF POPULATION FOR CITIES CHANGING FROM POLLUTED TO PURIFIED WATER SUPPLIES

	1907	1908	1909	1910
Columbus, Ohio..........	38 3	110.5	20.0	18 1
New Orleans, La........	55.5	33 1	28 4	31 5
Louisville, Ky...........	67.9	44 2	42.0	31 7
Pittsburgh, Pa..	130 8	46.6	24 0	27.8
Philadelphia, Pa	60.7	35 5	22 3	17.5

The filter plant for the City of Columbus, Ohio, was started in August, 1908; the high rate for this year was due to an epidemic in the early part of the year. The filter plants in New Orleans

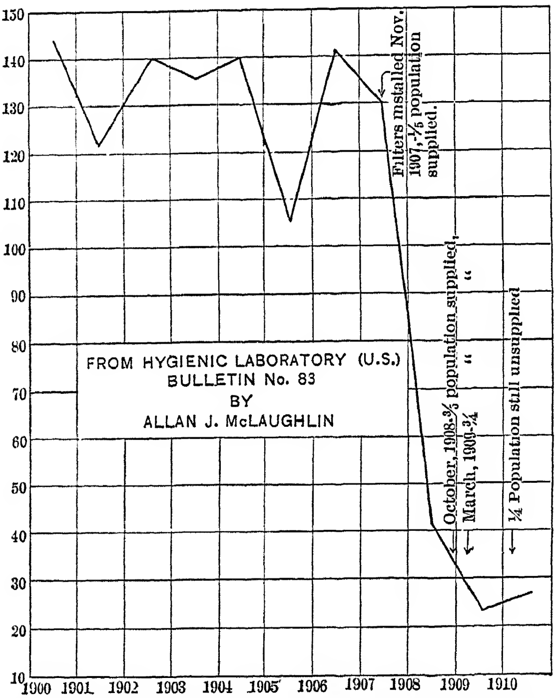

FIG. 3.—Typhoid death rate by years for city of Pittsburgh, Pa.

and Louisville were started in 1909. In Pittsburgh and Philadelphia filter plants were placed in operation in 1907 and 1908, respectively; but in neither city was the entire population supplied with the purified water.

Fig. 3 from Dr. Allan J. McLaughlin's paper on "Sewage Pollution of Interstate and International Waters," which was previously referred to, is of especial interest in this connection, in showing how pronounced a reduction in the death rate followed the introduction of even a limited volume of purer water.

From the typhoid fever statistics of Cincinnati, Ohio, a most convincing argument for the purification of a polluted water supply can be presented.

NUMBER OF CASES AND DEATHS FROM TYPHOID FEVER

Unfiltered water from old works					Filtered water from new works			
Year	1904	1905	1906	Total for 3 years	1908	1909	1910	Total for 3 years
Cases	1,646	746	1,940	4,332	235	218	183	636
Deaths	270	155	239	664	67	45	21	133

The figures for the year 1907 are omitted because water from both the old and the new works was supplied to the city.

If the above figures are expressed as cases and deaths per 100,000 of population, a better comparison may be made with other statistics.

NUMBER OF CASES AND DEATHS PER 100,000 OF POPULATION

For 3 years before introducing filtered water		For 3 years after intioducing filtered water		
	Average	. 1908	1909	1910
Cases................	417	67	62	50.0
Deaths..............	64	19	13	5.7
	Percentage reduction from the average.			
Cases................	....	84	85	88.0
Deaths..............	....	70	80	91.0

In a report on the purification of the Montreal water supply Messrs. Hering and Fuller present a table showing the effect of purification by filtration on the death rate from typhoid fever in a number of American cities.

The relation between impure water supplies and certain intestinal diseases other than typhoid fever, is more or less obscure.

Death Rates from Typhoid Fever per 100,000 Population in American Cities Using Filtered Water

City	Year plant was completed	Before filtration	After filtration	Before filtration	After filtration
		Years averaged		Death rate	
Sand filters					
Albany, N. Y.	1899	10	9	90	22
Lawrence, Mass.	1893	7	15	114	25
Pittsburgh, Pa.	1907	8	1	133	47[1]
Mechanical filters					
Binghamton, N. Y	1907	5	5	47	15
Cincinnati, Ohio	1908	4	1	50	16
Columbus, Ohio	1908	11	1	78	20
Paterson, N. J.	1902	5	7	32	10
Watertown, N. Y.	1904	5	5	100	38
York, Pa.	1899	2	8	76	22
Hoboken, N. J.	1905	7	4	19	14

Infant mortality from diarrhea and enteritis is probably both directly and indirectly the result of drinking impure water, although other causes contribute more frequently to death. About 85 per cent. of the deaths listed under "diarrhea and enteritis" in the United States census mortality statistics occur in children under 2 years of age. Dr. A. J. McLaughlin[2] in discussing this subject says:

"Instead of one disease designated under different names we are probably considering several diseases with common factors of transmission. Whatever the real relation between typhoid fever and enteritis or diarrhea of children may be, one fact is clear, the same causes operate to cause excessive prevalence of both. It is probable that cases of typhoid in children under 2 years in many cities are often incorrectly diagnosed as enteritis. It must be remembered, however, that the causative agent of bacillary dysentery is transmitted in the same way and by the same media as that of typhoid. There are too many cases of fatal illness in children under 2 years classed as diarrhea and enteritis, and an exhaustive investigation should be made to establish the real

[1] Including Allegheny, supplied with unfiltered water.

[2] Public Health and Marine Hospital Service. Hygienic Lab., *Bull.* 83, March, 1912.

cause of death in enteritis and diarrhea of children. Without such an investigation it is impossible to assign the real cause of the excessive child mortality from diarrhea and enteritis. In cities of less than 50,000 population without slums and which are not 'mill' towns an enteritis rate in children under 2 years above 100 deaths per 100,000 indicates prevalence of an acute intestinal disease preventable by the same measures that prevent typhoid fever. It is probable that in such cities proper enforcement of prophylactic measures against typhoid fever would reduce the enteritis rate below 40 deaths per 100,000. Enforcement of prophylactic measures would include the installation of pure water supplies and proper sewerage systems, coupled with a vigorous campaign against the insanitary outdoor privy and the equally dangerous shallow well."

Dr. McLaughlin gives the following table which emphasizes the complexity of the problem.

City	Typhoid death rate per 100,000, average for 10 years, 1900–1909	Character of water supply	Death rate enteritis, average for 5 years, 1904–1908	Remarks
Rochester, N. Y.	13.7	Good	89.5	Sanitary conditions good.
Syracuse, N. Y.	14.8	Good	105.5	
Albany, N. Y.	21.9	Good	80.0	
Binghamton, N. Y.	20.9	Good	104 7	
Utica, N. Y.	17.3	Good	133.7	Mill and factory towns; bad sanitary conditions.
Schenectady, N. Y.	22.4	Good	164.7	
Amsterdam, N. Y.	18.6	Good	150.7	
Yonkers, N. Y.	9.5	Good	207.7	
Cohoes, N. Y.	83.8	Polluted	170.9	Sanitary conditions, exclusive of water, good. Mill and factory towns.
Niagara Falls, N. Y.	129.1	Polluted	173.2	
Ogdensburg, N. Y.	148.5	Polluted	175.0	
Buffalo, N. Y.	27.0	Polluted	151.6	

The following mortality statistics taken from reports of the Health Department of Cincinnati, Ohio, point pretty plainly to the fact that a change from an impure water supply to a purified supply made a marked difference in the number of deaths from dysentery, diarrhea and enteritis in persons over 2 years of age.

The following figures show a reduction in deaths from dysentery for 3-year periods before and after the introduction of filtered water of 64.3 per cent. and for diarrhea and enteritis of 55.1 per cent.

Year	Unfiltered Water from Old Works. 1904	1905	1906	Total for 3 years	Filtered Water from New Works. 1908	1909	1910	Total for 3 years
Dysentery	27	21	22	70	9	11	5	25
Diarrhea and enteritis over 2 years of age	152	167	174	493	90	60	71	221

N. B. The year 1907 is omitted because water was supplied to the city both from the old works and from the new works.

THE MILLS-REINCKE PHENOMENON AND HAZEN'S THEOREM

The available evidence clearly proves that cholera, typhoid fever, dysentery and gastro-intestinal troubles are commonly transmitted by infected drinking water, and that there is reason to believe that other diseases are also conveyed in the same manner. The effect of purifying a polluted water supply, in decreasing the deaths from diseases other than cholera and typhoid, has been studied recently by W. T. Sedgwick and J. Scott MacNutt. They point out that in 1893–94, Messrs. Hiram F. Mills, C. E., of Lawrence, Mass., and Dr. J. J. Reincke of Hamburg, Germany, respectively, noted independently a decline in the general death rate of each of these cities as a result of improving their water supplies. Prof. Sedgwick and his associate have collected numerous mortality statistics in a paper[1] on this subject, and have termed the coincidence between a lowered death rate and a purified water supply as the "Mills-Reincke Phenomenon." In 1904 Mr. Allen Hazen in a paper read before the International Engineering Congress held in St. Louis, gave a quantitative expression for the phenomenon by stating that: "Where one death from typhoid fever has been avoided by the use of better water a certain number of deaths, probably two or three, from other causes have been avoided." Sedgwick and MacNutt have called this expression "Hazen's theorem," and have concluded from the studies which they have made, that the statement appears sound and conservative, but

[1] W. T. Sedgwick and J. Scott MacNutt: "On the Mills-Reincke Phenomenon and Hazen's Theorem Concerning the Decrease in Mortality from Diseases Other than Typhoid Fever Following the Purification of Public Water Supplies." *Jour. Infect. Diseases*, vol. 7, 1910.

not necessarily precise. The ratios they worked out varied widely. For example, at Hamburg, Germany, for every death less from typhoid fever after filtration, there were 15.8 deaths less from other causes; at Lawrence, Mass., the ratio was 1 to 4.4, at Lowell, Mass., 1 to 6.0, at Albany, N. Y., 1 to 4.1, and at Binghamton, N. Y., 1 to 1.5.

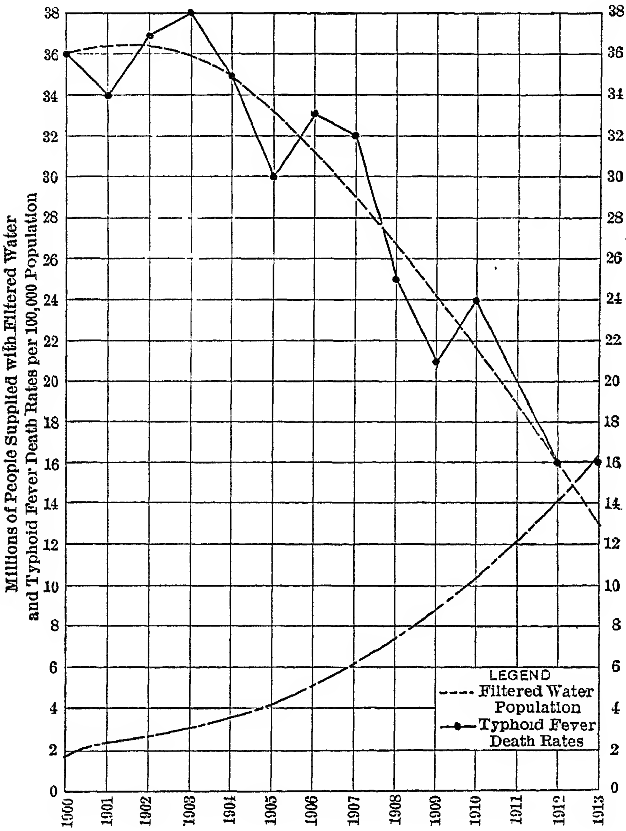

FIG. 4.—Growth of water filtration and decrease in typhoid fever death rate in the registration cities of the United States.

From the data which they compiled with respect to diseases of the respiratory organs, the evidence in many cases is striking and warrants a more extended study. Factors not specifically related to an improved water supply may have been potent in reducing the death rate in the cities studied. Greater knowledge of methods for the prevention of disease must have played its part

in diminishing mortality, and in extending the length of the average life. On the other hand, the real causes for many diseases are still unknown, and the probability that some of them can be transmitted through polluted drinking water is by no means unreasonable.

The elimination of disease germs from a water supply would be a direct cause for a decrease in the death rate, while an indirect cause could come from an increased vital resistance produced by the use of a purer drinking water. Probably both of these factors are instrumental in bringing about a lowered death rate.

The accompanying diagram (Fig. 4) from Mr. George A. Johnson's paper "The Typhoid Toll," in the *Journal of the American Water-works Association* for June, 1916, is a striking graphic presentation of the effect which water purification has had in reducing typhoid fever in the United States.

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CHAPTER V

OBJECTS AND METHODS OF WATER PURIFICATION

The purpose of purifying a drinking water is obviously to make it more pleasant and more wholesome to drink. The removal of suspended sediment and of vegetable coloring matter, in order to convert a dirty water into a clear and colorless one, commends itself if for no other reason than that the water becomes more acceptable to the senses of taste, smell and sight. If in addition to improving its appearance, it is possible to reduce materially or eliminate entirely the bacteria, some of which may be capable of producing disease, a second important reason for the purification of a drinking water can be advanced. By rendering a water hygienically safe for drinking purposes, sickness and death, as well as economic losses of no small magnitude are avoided. It is not difficult to show that a pure and wholesome water supply is a sanitary investment which pays in dollars and cents, as well as in health and happiness.

For industrial purposes many waters are entirely unfitted for use without purification. Usually the dissolved salts which produce hardness make a water unsuitable for use in steam boilers. The softening of hard waters is a method of purification which has long been practised, and which could be even more extensively employed today to the financial advantage of users of boiler scale-forming waters. In general, soft waters are more desirable for most industrial purposes. In such industries as brewing, distilling, starch and sugar manufacturing, the bacterial content of the water is of some importance, since certain organisms may produce fermentations which are undesirable. Waters containing iron in solution or suspension are especially objectionable for domestic purposes and for such industries as dyeing, paper making and bleaching.

General Methods of Purification.—The impurities in water which it is necessary to remove are in suspension and solution, and purification methods fall naturally into two classes in consequence. For the removal of matter in suspension two processes are employed, namely, sedimentation and filtration. These processes

occur normally in nature, but being uncontrolled produce results of varying degrees of excellence. When artificially controlled, sedimentation and filtration, properly carried out, are able to purify waters of very poor quality, and with a degree of certainty that have established these methods upon a scientific basis. Mineral matter in suspension can be entirely removed, as well as practically all of the organic matter, which latter includes the bacteria. Chemical reagents may or may not be employed to facilitate either or both of these processes. The chemical compounds used in this connection are mechanical agents exclusively, and merely assist and hasten the bringing together of finely divided suspended particles into larger masses, which will settle and filter out with greater ease.

For the removal of dissolved impurities, such as salts of lime and magnesia, chemical reagents are usually resorted to. A precipitation of the bases in an insoluble form, and their subsequent removal by sedimentation and filtration, constitute the usual methods of purification. In the case of easily oxidizable soluble salts, such as those of iron, aeration is effective in breaking down the ferrous compounds by the removal of dissolved carbon dioxide, and the introduction of sufficient oxygen to bring about the oxidation of the ferrous to the insoluble ferric compounds.

Filtration usually follows the process of sedimentation, and is carried out in various ways. Sand, gravel, coke, charcoal, coal, porous tile and sponges are some of the materials which have been and are still used as filtering media. Only the first two media, viz., sand and gravel, are commonly employed in large filtration plants.

Purification methods which do not fall under any of the abovementioned classes, are those of distillation and sterilization by chemical or physical agents. In the case of distillation all dissolved and suspended impurities are removed, as well as the bacteria being killed. It is obviously the most perfect method of purification, but not necessarily the most desirable. Its cost makes it prohibitive for all practical purposes so far as large supplies are concerned.

Disinfection and sterilization present the most advanced phases of water purification, and furnish methods of great practical value for supplementing other methods, as well as in some cases, furnishing themselves all the purification needed.

The action of these agents is primarily upon the readily oxidiz-

able matter which may be in a water, whether it be of organic or of mineral origin. The bacteria being a low form of plant life are thus attacked, and are either destroyed or their vitality so impaired that they no longer are able to reproduce themselves. The effect of these agents is not selective, except in so far as they react first with the more easily oxidized material. If not exhausted on the latter, they will react slowly upon the more resistant matter, but may fail to effect a complete oxidation or to hinder vital processes. Those bacteria which are capable of forming spores, and which may be in this stage, are sometimes not affected, on account of their well-known resistance to injurious conditions. The effect of these disinfectants upon the pathogenic forms is the same as upon the non-disease-producing bacteria which may be present in a water. If practically all the bacteria are destroyed, therefore, the water is rendered hygienically safe.

The agents used for disinfecting water are the hypochlorites of calcium and sodium, pure chlorine gas, ozone, copper sulphate, and the rays of ultra-violet light. Other chemical compounds have been suggested and experimented with, and the use of the electric current has frequently been tried. None of these agents is of much practical value excepting chlorine and its compounds in the form of the hypochlorites, ozone gas and the ultra-violet light. All of the chemical disinfectants are used in very minute amounts in water-purification work; nevertheless, they are extremely effective when properly applied. The relative merits of these various agents will be discussed in a later chapter.

The use of copper sulphate as an agent for killing off growths of algæ and diatoms in water supplies is of practical importance, and of much value. The troublesome character of these growths in the operation of water-works, and even of purification plants will be considered in detail later.

Statistics of Purification Plants.—Fifty years ago in the United States no plants for the purification of a public water supply were in existence. In Europe plants which clarified the water were in use, and incidentally effected a certain degree of bacterial purification. They were operated upon an empirical basis and with no true understanding of the scientific principles underlying the art. In fact, the modern theory of the cause for many diseases was unknown, and the part played by minute vegetable and animal organisms in the dissemination of disease was undreamed of. By the wonderful investigations and deductions of Pasteur

and Koch, the relation between polluted waters and the transmission of certain diseases became apparent and furnished a rational basis upon which methods of purification could be constructed.

According to statistics first compiled by Mr. Allen Hazen for the International Engineering Congress held in St. Louis in 1905, and brought up to 1910 by Mr. Geo. C. Whipple in a paper read before the Congress of Technology held in Boston in 1911, over 10,000,000 people in the United States are now supplied with water purified by filtration. Mr. Whipple presents some tables which clearly indicate how rapid the growth in the construction of purification plants has been.

Populations Supplied with Filtered Water at Different Dates

Year	Total urban population in U. S., places of more than 2,500 inhabitants	Population supplied with filtered water			Per cent. of urban population supplied
		Sand filters	Mechanical filters	Total	
1870	....	None	None	None	0 00
1880	13,300,000	30,000	......	30,000	0.23
1890	21,400,000	35,000	275,000	310,000	1.45
1900	29,500,000	360,000	1,500,000	1,860,000	6 30
1910	38,350,000	3,883,221	6,922,361	10,805,582	28.20

In 1870 water supplies in the United States which were purified by filtration were practically unknown, while today probably 30 per cent. at least of communities of more than 2,500 inhabitants are drinking filtered water. Mr. Whipple estimates that:

"If the cities of more than 25,000 inhabitants are considered alone, it will be found that our 228 cities have a total population of 28,508,000. Of these, about 8,098,000 are supplied with water that does not need filtration, or at least will not for a long time. This leaves about 20,311,000 people that are using water from sources subject to contamination. Of these 8,402,000 or 42 per cent. are adequately protected by the filtration of the water. Filters are under construction or have been authorized for 648,000 more, thus raising the percentage to 45 per cent. Filters have been officially recommended for 3,541,000; 7,720,000 people are still using water of questionable quality, although in some of these cases filtration has been seriously considered by sanitarians."

A summary of statistics of population supplied with filtered water in the United States is given by Mr. Whipple in another table, which is well worth quoting.

SUMMARY OF STATISTICS OF POPULATION SUPPLIED BY FILTERED WATER IN THE UNITED STATES

Cities with populations	Number of cities	Total population, census, 1910	Filters in service		Filters under construction and authorized		Filters recommended officially	Filters needed now or probably will be	Filters probably not needed for a long time	Filters not needed with present water supply
			Sand	Mechanical	Sand	Mechanical				
Over 1,000,000	3	8,501,174	1,601,008	104,000	.....	......	2,850,600	2,185,283	1,760,283	....
200,000 to 1,000,000........	25	8,982,345	1,452,331	1,654,495	859,893	......	....	2,971,817	1,680,611	363,198
100,000 to 200,000..........	22	2,819,528	125,253	893,036	. ..	... ..	112,571	600,719	850,550	237,399
50,000 to 100,000...........	59	4,178,915	398,103	796,264	.....	89,336	147,325	952,484	1,041,465	753,938
25,000 to 50,000	119	4,026,045	176,911	1,300,411	.....	......	128,993	1,009,374	629,588	780,768
Total for cities over 25,000...	228	28,508,007	3,753,606	4,748,206	558,485	89,336	3,540,897	7,719,677	5,962,497	2,135,303
2,500 to 25,000 estimated[1]...	...	9,841,993	129,615	2,174,155						
Total for places over 2,500[1] .	...	38,350,000	3,883,221	5,922,361						

[1] Census returns not available when table was compiled.

A recent compilation of the rapid sand or mechanical filter plants now in operation in America, together with their daily capacities is shown in the following table:

RAPID SAND OR MECHANICAL FILTER PLANTS

		Having daily capacities of		
	10 M G. or over	3 M G to 10 M G.	Less than 3 M G	Total
Number of plants.	29	112	365	506
		Millions of gallons		
Total daily capacity as grouped.	892	520	332	1,844

Mr. George A. Johnson, in his recent paper on the "Typhoid Toll,"[1] estimates that the total population of the cities of the United States making returns of vital statistics is 34,230,000. Filtered water was supplied to 48 per cent. of the population of these cities in 1913, or in other words to a population of 16,500,000 persons. This latter figure represents 17 per cent. of the total estimated population of the United States.

The purification of water supplies by disinfection with calcium hypochlorite, either continuously or intermittently, has gone forward by leaps and bounds within the past 5 years. Today it is estimated "that 300 to 350 cities in the United States alone use this process."[2] The use of disinfecting agents in water supplies is also increasing in Europe.

The investment represented by purification plants mounts into many millions of dollars, but the conservation of life effected by them, measured in dollars, is very many times more than their first cost. Efficiency of these plants is of the utmost importance both from the hygienic and economic standpoint, and is becoming more and more appreciated by communities owning them. To the technical problem of purification much study has been given in the past, and is still being given at present; but the art as a whole is on a thoroughly scientific and practical basis, as the beneficial results of water purification amply testify.

[1] *Jour. Am. Water-works Assn.*, June, 1916.

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CHAPTER VI

SEDIMENTATION

The removal of suspended mineral and organic matter from water supplies is most easily accomplished by settling in reservoirs. The clarification and purification effected by settling in this manner is usually spoken of as plain sedimentation. By the addition of certain chemical compounds to naturally turbid waters, an artificial flocculation or coagulation of the suspended particles is produced whereby they settle out more rapidly. This process is termed "sedimentation after coagulation" or "precipitation with chemicals."

PLAIN SEDIMENTATION

Turbid surface waters flowing into impounding reservoirs become partially clarified in passing slowly through or in standing undisturbed in them. This result is produced because the velocity of flow of the water, which has maintained these minute particles in suspension, is lessened or reduced to practically zero. The reduction in the velocity of flow permits the sediment to gravitate toward the bottom of the reservoir, and to be finally deposited thereon. Provided the wind does not set up surface currents, nor temperature changes produce vertical currents, the sooner will the deposition of the sediment be effected. For obvious reasons a perfectly quiescent condition of the water can not be produced, and complete clarification, therefore, is not usually practicable.

The ponds and lakes are, as a rule, natural settling reservoirs, in so far as the turbid surface waters which may flow into them are concerned. The water in them is, therefore, usually quite clear. The factors tending to disturb the water and to retard settlement are of course active at times.

Artificial settling reservoirs and coagulation basins form a most important part of purification plants treating clay-bearing waters. As a preliminary treatment for filtration they are absolutely essential for efficiently and economically handling these turbid waters. They may be also of much assistance in

purifying waters which are only turbid during portions of the year. In water softening, settling basins are essential to obtaining a well-clarified effluent.

Theory of Sedimentation.—The very practical importance of sedimentation in water purification has caused the conditions which influence the phenomena to be carefully studied. Much of the experimental work has of necessity been empirical in character, because of the number of factors which are involved. Some investigators, however, have approached the problem from a theoretical standpoint, and many of their deductions are valuable and suggestive. The early work of Brewer, Durham, Hunt, Barus and Seddon is of interest in connection with this general subject. The more recent paper of Mr. Allen Hazen,[1] however, has a more practical bearing on sedimentation, although discussing in a theoretical manner certain important factors.

From the simple assumption, "that whenever a particle of suspended matter hits the bottom, it remains where it strikes, and is never carried forward on the bottom, or picked up again; second, that all the sediment in the water is of the same hydraulic value, that is to say that every particle settles through the water at the same rate as every other particle," Mr. Hazen proceeds to develop mathematically fifteen special propositions. He discusses the velocity with which particles of sediment settle through still water, and the effect of temperature, flocculation and coagulation upon the rate of settlement.

The minute size of the particles, their very slow subsiding velocities, and the mixing action produced by surface and vertical currents, all tend to maintain the finest particles in practically constant suspension. Mr. Hazen gives a table of the subsiding velocities of various-sized particles which is reproduced on the following page.

Mr. Hazen's résumé of the deductions and conclusions at the close of the paper is clearly and concisely stated as follows:

Résumé

"The fundamental proposition, in clearing water by sedimentation, seems to be that every particle of sediment moves downward through the water at a velocity depending upon its size and weight and upon the viscosity of the water. Particles of sediment are generally so far apart that they do not influence each other; and, while there is no doubt

[1] *Trans.* Am. Soc. C. E., vol. 53, p. 45, 1904.

VELOCITIES AT WHICH PARTICLES OF SEDIMENT FALL IN STILL WATER

Diameter of particles in millimeters	Hydraulic value in millimeters per second 10°C. = 50°Fahr	Remarks
1.0000	100 0	Experiments by the writer.
0.8000	83 0	Experiments by the writer.
0 6000	63 0	Experiments by the writer.
0 5000	53 0	Experiments by the writer.
0 4000	42 0	Experiments by the writer.
0 3000	32.0	Experiments by the writer.
0.2000	21 0	Experiments by the writer.
0 1500	15.0	Experiments by the writer.
0.1000	8.0	Experiments by the writer.
0.0800	6 0	Interpolated from connecting curve.
0.0600	3.8	Interpolated from connecting curve.
0 0500	2 9	Interpolated from connecting curve.
0 0400	2.1	Interpolated from connecting curve.
0 0300	1.3	Interpolated from connecting curve.
0.0200	0 62	Wiley's formula.
0 0150	0 35	Wiley's formula.
0.0100	0 154	Wiley's formula.
0.0080	0 098	Wiley's formula.
0.0060	0.055	Wiley's formula.
0.0050	0 0385	Wiley's formula.
0.0040	0 0247	Wiley's formula.
0.0030	0 0138	Wiley's formula.
0.0020	0.0062	Wiley's formula.
0.0015	0 0035	Wiley's formula.
0.0010	0.00154	Wiley's formula.
0 0001	0.0000154	Wiley's formula.

NOTE.—These values are not given as being precise, but they are believed to be sufficiently accurate for the purpose of this discussion.

that they do sometimes collect in groups and thus change the conditions, it seems to be generally true that each particle will settle as if no other particle were present.

"If the water in a basin were absolutely quiet there would be a regular sequence of clearing beginning at the top. The coarsest particles would go down fastest, but at any given point there would be a gradual clearing, and this clearing would take place most rapidly at the top, and, after longer intervals, at lower points in the basin.

"Seddon started out with this theory, but found it to be not in accordance with the facts. His observation showed that while the amount of sediment in the water in the top was a little less than in the water in the bottom, the distribution was nearly equal throughout the mass, a condition of affairs inconsistent with the theory. He

accounted for this distribution of sediment by the constant mixing of the water from top to bottom, and to the sustaining power of vortex motions in the water. These motions he thought arose from the internal motion of the water at the time of entrance, and from wind, and from temperature changes.

"The writer has taken Seddon's development of the case as his starting point, and has carried the discussion further. He believes that while the internal motions keep the water mixed, and with nearly the same density of sediment from top to bottom, the tendency of the particles of sediment to settle is nevertheless an unbalanced force always acting to take the particles to the bottom, and the number of particles that hit the bottom in a given time is proportional, first, to the velocity at which the individual particles settle, and second, to the density of sediment in the water immediately above the bottom.

"With these fundamental relations in mind, it is easy to compute and to express by simple formulas the proportions of particles of sediment of a given hydraulic value which will hit the bottom under given conditions and which, therefore, presumably will be removed.

"The fundamental propositions may be very concisely expressed. They are: First, that the results obtained are dependent upon the area of bottom surface exposed to receive sediment, and that they are entirely independent of the depth of the basin; and second, that the best results are obtained when the basins are arranged so that the incoming water containing the maximum quantity of sediment is kept from mixing with water which is partially clarified. In other words, the best results are obtained where any given lot of water goes through the basin with the least mixing with the water which enters after it. This is practically accomplished by dividing the basins into consecutive apartments by baffles or otherwise.

"Thus far, the discussion is easy and apparently certain. The next step is a more difficult one. It relates to bottom velocities, and has to do with the question whether these velocities are such as to allow the particles to remain on the bottom when they get there, or whether they will be taken up again and be kept in motion with the body of the water. This is a point upon which further experimental data are needed. The problem of securing such data seems to be difficult. The observations must be made at the bottom of a layer of liquid of considerable thickness, where the conditions of observation are not favorable. The observations, further, must be made on very low velocities and on particles so small as to be practically microscopic.

"Whatever view may be taken of the second part of the problem, or whatever researches upon it may show, the arrangements of basins most favorable to taking particles to the bottom should stand."

Coagulation of Sediment.—The collection of minute particles of suspended matter into larger aggregates is a process which appears to occur only to a limited extent in natural waters, although it may be artificially produced by the addition of certain chemical compounds. Since the deposition of these finely divided particles greatly facilitates sedimentation and subsequent filtration, when the latter forms a part of the purification process, the phenomenon is of much practical interest and importance in water purification.

Colloidal Character of Sediment.—The analogy between the physical properties of liquid suspensions of certain chemical compounds, and those of turbid clay-bearing waters is so marked that some description of their properties will be of interest. In a paper[1] published some years ago by the author attention was called to Dr. A. A. Noyes'[2] classification of colloidal mixtures, in which he defined non-viscous, non-gelatinizing, but readily coagulable mixtures as "colloidal suspensions." He regarded them as really suspensions of minute particles and not true solutions. He differentiates further by designating mixtures in which the particles may be visible under the microscope as "microscopic suspensions," reserving the term "colloidal suspensions" for those containing particles beyond the limit of microscopic visibility.

Physical and Electrical Properties.—Natural waters furnish many examples of colloidal suspensions. Most turbid waters in their natural condition may be considered as mixtures of "microscopic and colloidal suspensions." A beam of light passed through Ohio River water, even after the water has stood for many weeks, is plainly visible, in the same manner as when a sunbeam passes through dusty air. This is a fairly good proof of the presence of minute particles in suspension, which reflect the light from their surfaces. Colloidal suspensions of gold and arsenious sulphide artificially prepared act in a similar manner to a ray of light.

Another property of colloidal and microscopic suspensions, apparently depending on the presence of electric charges upon them, is seen in the migration of the colloidal particles by the passage of an electric current through the mixture. Thus the particles of a colloidal suspension of ferric hydroxide or aluminum

[1] "The Coagulation and Precipitation of Impurities in Water Purification." *Eng. Record,* vol. 51, May 13, 1905.

[2] *Jour. Am. Chem. Soc.*, vol. 27, No. 2.

hydroxide migrate with the positive current toward the cathode, while kaolin and other similar colloidal or microscopic particles migrate with the negative current toward the anode. The colloidal suspension of clay particles characteristic of turbid Ohio River water act in a precisely similar manner to those of kaolin. Experiments by the author with slightly turbid Ohio River water produced an average rate of travel of the particles toward the anode of 0.29 cm. per hour for a potential gradient of 4 volts per centimeter. These results are of the same order of magnitude as were obtained by Whitney and Blake for the migration of particles in a colloidal suspension of silicic acid.[1] Silica doubtless formed the larger proportion of the particles in the sample of Ohio River water with which the above experiments were made. The clearing of the water at the cathode and the movement of the particles toward the anode indicated them to be negatively charged.

Coagulation with Chemicals.—Turning now to another important property of colloidal suspensions in general, viz., their coagulation, we find that turbid natural waters possess similar properties to colloidal suspensions artificially prepared. Non-electrolytes do not have the power of coagulating colloidal suspensions; but all electrolytes will do so with a proper degree of concentration of the latter, and a sufficient length of time. Acids, bases and salts, therefore, will coagulate colloidal suspensions, all being more or less dissociated in aqueous solution and capable of conveying an electric current. In a similar manner hydrochloric acid, caustic soda, caustic lime or an ordinary salt solution will each, if of the proper concentration, coagulate the colloidal clay of a naturally turbid water like that in the Ohio River. It should always be borne in mind in dealing with natural waters that we are working with extremely dilute solutions of salts, even in those waters which we commonly speak of as high in soluble compounds. In other words the concentration of the electrolytes is very low.

In a number of modern purification plants in which lime is used to soften the water, in addition to the employment of sulphate of iron or aluminum sulphate to effect clarification, the action of the caustic alkali is virtually that of a coagulant, especially if the lime is added in sufficient amounts to produce a slightly caustic condition. Some quite extensive experiments undertaken on a large scale some years ago, clearly proved that by rendering the Ohio

[1] *Jour. Amer. Chem. Soc.*, October, 1904.

River water caustically alkaline with lime, the suspended clay could be coagulated, and could be subsequently removed by rapid filtration through sand. The explanation of the phenomenon seems to be that the introduction of the base calcium hydrate in excess, furnished an electrolyte of sufficient concentration in the water to effect coagulation of the colloidal clay.

The similarity between artificially prepared colloidal suspensions and the very small suspended particles characteristic of turbid waters as above described, seems to warrant the classification of the latter mixtures with the former. If we seek further analogies in the co-precipitation or absorption by colloids of other substances in solution and suspension with them when these colloids are coagulated, the true character of turbid waters seems even more apparent.

When a turbid water containing carbonates of lime and magnesia in solution is treated with sulphate of alumina, for example, a reaction results setting free aluminum hydroxide. The liberated aluminum hydroxide absorbs or mechanically traps the finely divided material in suspension, and in the course of time the finer particles come together to form larger particles which settle out readily. This process of coagulation of the colloidal clay suspension and co-precipitation of the larger suspended particles, appear to be started by the coagulation of the aluminum hydroxide. It is evident, as Dr. Noyes points out in the paper referred to, "that the mechanism of this coagulation is not yet understood," although it appears, "that it is the ion with a charge opposite to that of the colloid particles that is mainly responsible for their coagulation." It is also probable that it is the electric charge upon the particle which tends to hold it in suspension.

Time Required for Coagulation.—The time factor in the coagulation of turbid waters is an important one. The length of time required to bring about the proper degree of coagulation varies with different waters. It is apparently influenced by the character of the suspended colloids, by the kind and amount of salts in solution, by the quantity of the coagulating chemical applied, by the temperature of the water, and by the agitation to which the mixture is subjected. The presence of much organic matter may retard the formation of the floc. An ordinary clay suspension is usually quickly coagulated. Complete flocculation and partial clarification may take place under favorable conditions in as short a time as 10 or 15 min.; on the other hand, the

author has seen a water which was not affected at the end of 4 hrs. This water had only a slight turbidity. Two hours after applying the coagulating chemical to this water it could be passed through a sand filter without removing the suspended matter or all of the coagulant. The large amount of soluble organic matter present in the water evidently produced a colloidal suspension with the aluminum hydroxide, formed by the decomposition of the aluminum sulphate which was added, and was only slowly precipitated.

Under ordinary conditions a period of 3 to 5 hrs. is sufficient to properly coagulate and settle a turbid water. Of course a proper arrangement of basins, inlets, outlets and baffles is requisite for bringing about effective coagulation and clarification. The construction of typical coagulation and settling basins will be described in a succeeding chapter.

Loss of Chemical by Adsorption.—Another phenomenon common to the coagulation of turbid waters is the adsorption of the applied chemical compound by the flocculent material produced by the reaction and the associated suspended matter. The particular significance of this fact in water purification was pointed out by Mr. George W. Fuller in his report on the experiments on Ohio River water made at Louisville, Ky., some years ago. While adsorption or concentration of a liquid on the contact-surface of the particles is not alone peculiar to colloids, but is characteristic of all solid precipitates, it has a practical bearing on coagulation in that there is some loss of chemical on account of this property. The amount of chemical thus lost is probably proportional to the surface area of the particles of the precipitate, and is a function of the nature of the solid and dissolved bodies, and of the concentration of the latter.

Natural Colloids.—The "schmutzdecke," which forms on the top of slow sand filters, is doubtless a true colloid, and by its adsorptive power removes the organic and inorganic suspended matter. The difference between the appearance of the sand in the beds of slow sand filters receiving water carrying considerable dissolved and suspended organic matter, and those which receive water holding more or less clay in suspension, as pointed out by Mr. George W. Fuller in his discussion of Mr. Allen Hazen's paper "On Sedimentation," previously quoted from, may possibly be explained by a difference in the character of the colloids produced or carried by the two types of water. The colloid formed

by the soluble and suspended organic matter in a water has probably the characteristic of a true colloid, *i.e.*, a viscous, gelatinizing compound not coagulated by salts, like gelatine for example. On the other hand clay-bearing waters produce or carry a colloid which is non-viscous and non-gelatinizing, but which may be readily coagulated by compounds like aluminum sulphate or sulphate of iron. Bacterial activity very likely aids in the formation of the natural scums found on slow sand filters.

In the absence of any artificially applied coagulant like aluminum hydrate the "granular appearance" of sand beds receiving clay-bearing waters indicates the formation of a colloid whose ability to prevent the passage of finely divided suspended matter is limited. The colloidal particles pass deeper into the sand than would a true gelatinous colloid, and their power of adsorption is soon exhausted. The "ripening" of the sand beds of both slow sand and rapid sand filters, is, therefore, probably a process of the formation and deposition of colloidal particles upon the sand grains, by which their adsorptive power is slowly increased.

It is, therefore, in the character of the colloid formed that one must seek for the explanation of the phenomena connected with the coagulation and filtration of natural waters. It is evident that much more light is needed on a great many of these problems, but that some of them are associated in some way with the colloidal state of the clay, of the organic matter and of the artificially applied coagulants, is self-evident. While the above points may at first sight appear to be only of scientific interest, they are in reality intimately associated with the most efficient design of purification plants. The size of coagulation basins, the mixing and the agitation of the raw water with the coagulant, the ability to vary the period of coagulation for any given water, the best chemical compounds to use for the proper coagulation of the suspended matter in the water, the advisability of providing for plain sedimentation before attempting coagulation and the size and character of the sand grains composing the filter bed are all factors entering into the practical design of water-purification plants. The poor results often obtained in the operation of plants is quite as frequently the result of improper design as of faulty methods of handling the plant.

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CHAPTER VII

TYPES OF SETTLING RESERVOIRS AND COAGULATION BASINS

The design of reservoirs and basins for efficient sedimentation has received considerable attention in water-purification work, but not as much as the subject deserves. The complexity of the problem has not always been appreciated, and too frequently this portion of the plant has been made to accommodate itself to other features of the design instead of being properly coördinated with them.

A certain amount of flexibility in the design of these reservoirs is quite possible without materially sacrificing efficiency. Nevertheless, it is the author's opinion that much more of the work of clarification should be thrown on this part of the purification plant than is now the practice. They are by far the least complicated portions of the works, and the least expensive to operate. Their tendency to make more uniform the operation of the filters strongly commends them to the operators of such plants. It is believed that in designing settling reservoirs of all kinds more consideration should be given to obtaining efficient sedimentation and to making them as large as good engineering practice will permit, due regard being had for other portions of the plant, in order to produce a well-balanced design.

Impounding Reservoirs.—Impounding reservoirs built primarily for storing water are, of course, to be considered from a somewhat different standpoint than those especially designed for sedimentation purposes. The depositing of suspended sediment in storage reservoirs is an incident rather than the object of their operation. However, considerable purification is effected in them, and the influences which lead to such results will be discussed in another section.

Settling Reservoirs.—Storage of turbid waters for purposes of plain sedimentation, in reservoirs especially constructed for this purpose, is undoubtedly good practice, but is not always provided for. Usually such supplies are drawn from turbid streams in which the water may at times be loaded with sediment. Provided the reservoirs are relatively large in proportion to the con-

sumption of water, quite a number of days of settlement may be possible before the water is drawn off. The relative positions of the inlet and outlet of these reservoirs is obviously of great importance in even approximating theoretical displacement of the water if they are used continuously. The tendency of the water currents to seek the shortest path between the inlet and outlet, and thereby render more or less ineffective certain parts of the reservoir, is a commonly observed condition. Baffling undoubtedly has the effect of breaking up "short-circuiting currents of water," but is not always employed in plain sedimentation basins.

Cleaning out the sediment deposited in this class of reservoirs is greatly facilitated by smooth and hard linings to the reservoirs, by sufficient slope to the sides and bottom and by an adequate system of gutters and drains. Proper facilities for flushing out the mud by streams of water under pressure, or by scraping the mud to the gutters assisted by a flow of water which is not under pressure, are necessary in all well-designed basins.

In all settling basins the sand and heavier portions of the silt will be found deposited close to the inlet. In some cases the amount of sediment dropped near the inlet is very great, especially where the turbid stream supplying the water carries a good deal of sand and silt, and where the pumping from the stream is practically continuous.

Cincinnati Storage and Settling Reservoirs.—Combined storage and settling reservoirs are in operation at Cincinnati, Ohio, (Fig. 5) in which Ohio River water undergoes plain sedimentation before being coagulated with chemicals and filtered. These two reservoirs hold approximately 392,000,000 gal. of water, and were designed with the idea of their being operated on the fill-and-draw plan. This method of operation, however, has never been followed. The water flows continuously through them in parallel. The inlets to the reservoirs are between 500 and 600 ft. from the outlets. The latter consist of movable pipes (Figs. 6 and 7) with their mouths held about 4 ft. under the surface of the water by means of floats. The water is thus continuously skimmed from the surface. The depth of water in the reservoirs varies from 35 to 50 ft.

The irregular shape of these reservoirs (Fig. 8) is accounted for by the desire in construction to make the excavations equal the embankments as nearly as possible. As it was intended that

they should be operated by first filling with water, then allowing the latter to stand and deposit its sediment for a day or so, and finally drawing off the settled water, it was of no particular consequence if the inlet was close to the outlet. Since they are not

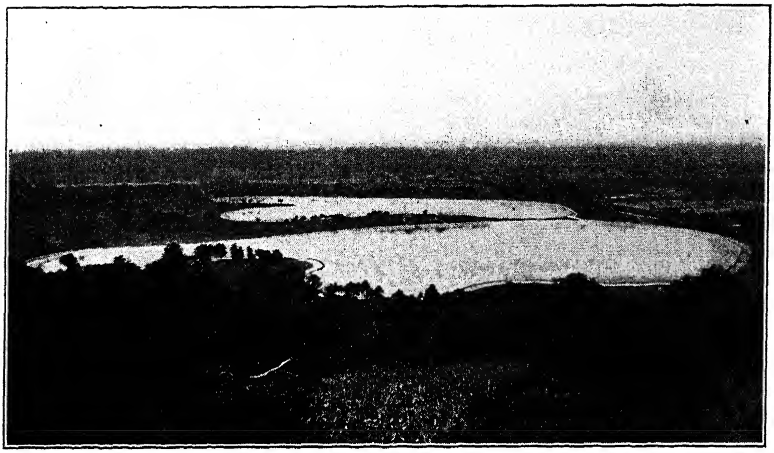


FIG. 5.—Cincinnati settling reservoirs.

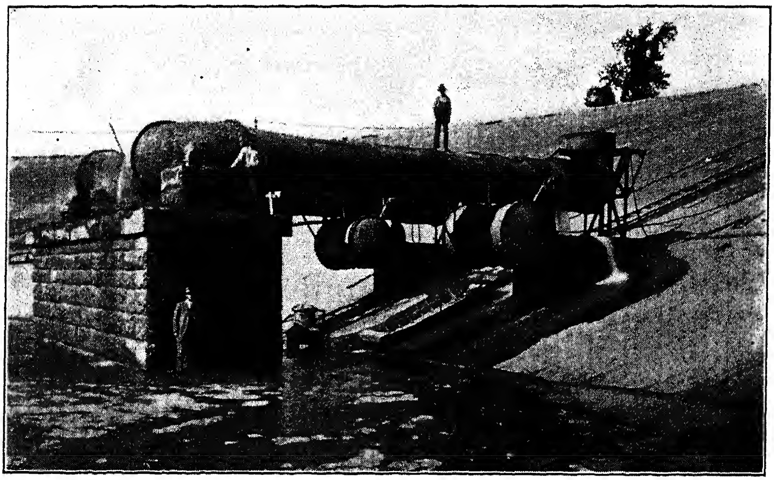


FIG. 6.—Cincinnati settling reservoirs, effluent float tubes.

utilized in this manner, but are operated continuously, their efficiency is undoubtedly somewhat reduced because of their irregular outline, and the short distance between the inlet and outlet. The apparently dead spaces in these reservoirs, however,

are by no means useless, since experience shows that diffusion of the sediment causes a much more uniform deposition of the latter than would be supposed. The depth of mud in the lobes of these

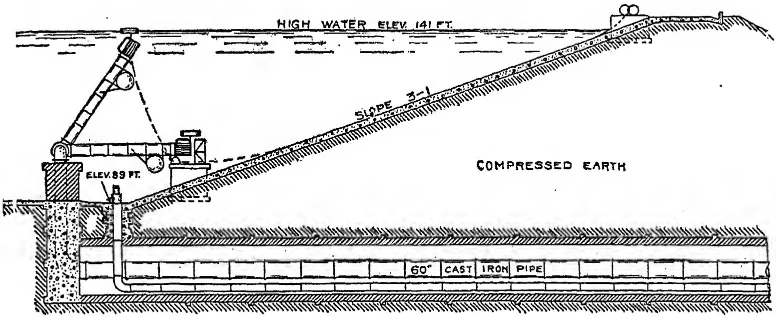


Fig. 7.—Cincinnati settling reservoirs, profile showing float tubes.

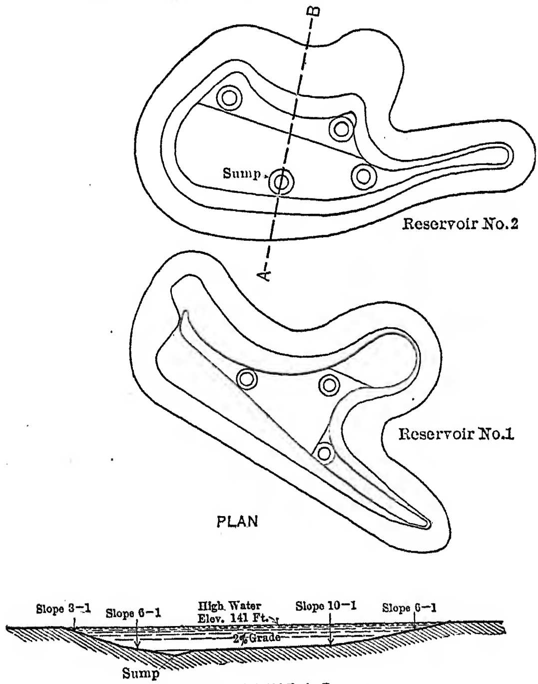


Fig. 8.—Cincinnati settling reservoirs, plan and cross section.

reservoirs will probably run from 35 to 40 per cent. of the average depth in the portions of the reservoirs where more active sedimentation is in progress.

Louisville Settling Reservoirs.—The two Crescent Hill reservoirs at Louisville, Ky., perform a similar service to those at Cincinnati. They cover an area of 750,000 sq. ft., and hold, when filled to a depth of 20 ft., a little over 100,000,000 gal. of water. Originally the water as it was pumped from the Ohio River was delivered into the gate house at the end of the division wall separating these two basins. It passed over a weir above the 20-ft. level into the first basin, and was drawn off at a point diagonally opposite at the bottom through a conduit 5 ft. in diameter. Through this conduit the water passed to the corresponding corner of the second basin, where it entered at the bottom. It was drawn off at the diagnonally opposite corner through the gate house.

Delivering the muddy water at the surface of these reservoirs and drawing off at the bottom has been recently changed, so that the water now enters from the gate house at the bottom of the first basin, and is withdrawn over a weir tower built to enclose the inlet end of the conduit. After passsing through the latter to the bottom of the second basin, it is skimmed off at the top of this basin at the gate house from which it flows to the coagulation basins.

Mr. George W. Fuller observed, when these reservoirs as originally arranged, were drawn off to be cleaned, that with the exception of the coarser material piled up near the inlet, the depth of sediment was substantially uniform over the entire bottom. Mr. Allen Hazen has noted the same condition in other reservoirs, and accounts for it by the mixing action by which water in all parts of the reservoir is made substantially of the same quality.

New Orleans Grit Reservoirs.—For removing the heavy silt and sand of a normally turbid water, relatively small grit reservoirs have been used. In the New Orleans purification plant two such reservoirs are in service. Each reservoir is 75 by 150 ft. in area, and has outer walls 20 ft. high. These reservoirs are provided with a center baffle which causes the water to travel up one side of the reservoir and down the opposite side to the outlet. At the normal capacity of the plant a sedimentation period of about 1 hr. is obtained with one basin in service. The heaviest sediment settles nearest the inlet, and grows much lighter as the outlet is approached (Fig. 9).

Albany Settling Reservoir.—The settling basin designed to be used in conjunction with the slow sand filtration plant at Albany,

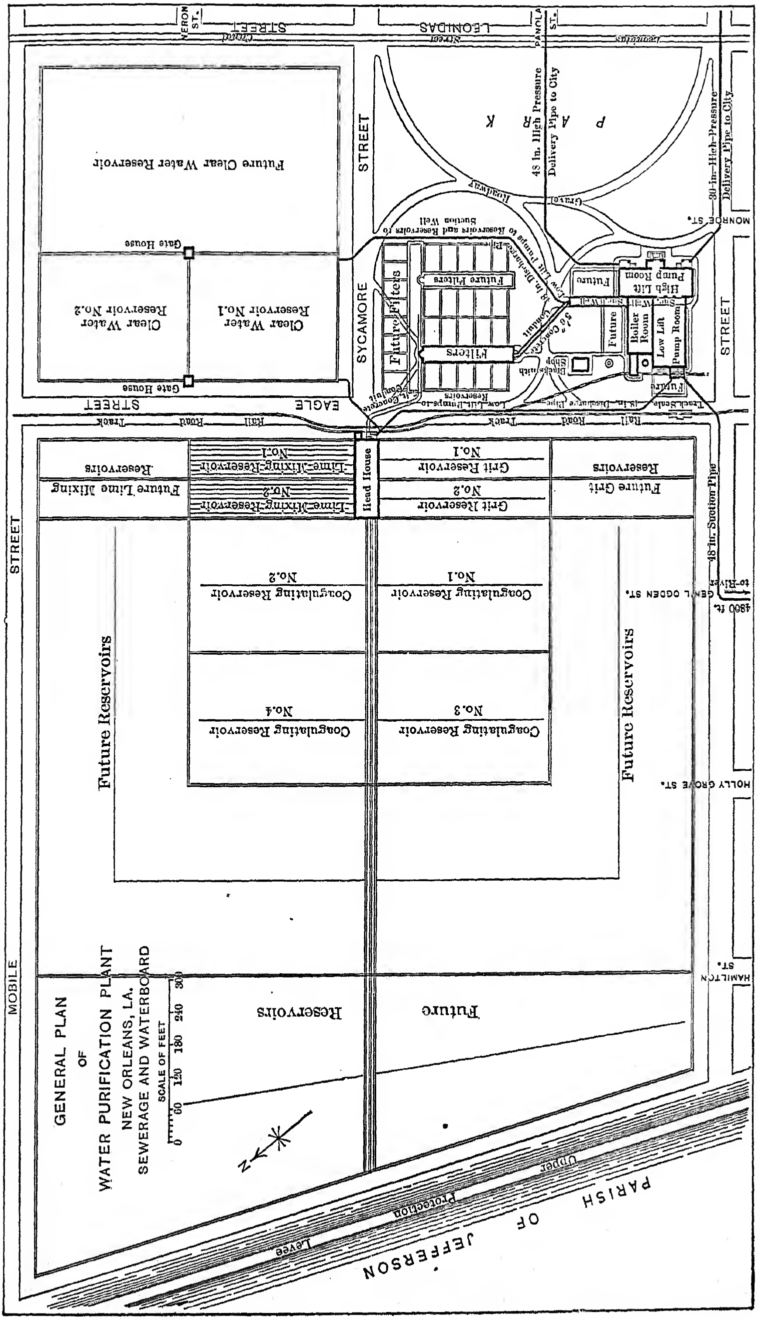


FIG. 9.—New Orleans water purification plant, general plan.

N. Y., was intended to be operated continuously. The basin has an area of 228,000 sq. ft. and a depth of 9 ft. It holds 14,600,000 gal. of water. Provision was made to allow the water entering the reservoir to discharge through 11 inlet pipes, which rise 4 ft. above the water line. By discharging into the basin in this manner aeration of the water was produced. The water was withdrawn through 11 outlets.

At the present time the water is not aerated as it enters the reservoir through two 18-in. perpendicular and one 36-in. horizontal inlets. At the present rate of consumption a 17-hr. period of subsidence is obtained.

COAGULATION BASINS

The bringing together of finely divided particles of suspended sediment in natural waters by means of the action of certain chemical compounds is termed coagulation. Basins, in which this action may take place and the flocculated sediment settle out, are commonly provided in connection with rapid sand filter plants. They may also form a part of slow sand filter plants where the sediment in the water settles out slowly and is liable to clog the sand in the filters.

Mixing Channels.—The introduction and uniform distribution in the water to be treated of chemical solutions of the strengths usually employed in water purfication offers some mechanical difficulties. Since the efficiency of the coagulating compound depends upon the formation of a large and well-defined floc, the size and character of which is directly influenced by the temperature of the water and the nature of the salts and colloids in solution and suspension, a thorough mixing of the water with the solution is of the utmost importance.

By sending the water through relatively narrow channels at a velocity of 1.5 to 2.0 ft. per second, and allowing the travel through these channels to take about 1 hr., a thorough mixing and a complete coagulation is effected, which produces rapid sedimentation in the coagulation basins into which the water discharges. The eddies formed in the channels by reversing the direction of flow or by baffles or by projecting portions of the concrete construction in the channels, assist materially in bringing about a thorough mixing.

In New Orleans mixing channels are provided for applying a 5 per cent. milk of lime in the manner described above. In this

plant duplicate reservoirs 75 by 320 ft. each with outer walls 19 ft. high, are divided into 16 rectangular double-decked passages with a cross-sectional area of over 60 sq. ft. each, and an aggregate length of about 5,120 ft. When passing 40,000,000 gal. of water in 24 hrs., the flow of the water through the channels requires about 1 hr.

Iron sulphate for coagulation purposes, and soda ash for reducing the permanent hardness of the water, may be both introduced in these channels, but provision is also made for applying a coagulant at various points in the flow of the water through the coagulation basins proper.

Obviously, mixing channels may become settling chambers unless the velocity of flow is sufficient to retain the flocculated sediment in suspension. This does take place, but corrects itself by reducing the cross-section of the channel by deposition of sediment until the velocity is sufficient to maintain a scouring action through the passage. On the other hand, too great a velocity may break up the floc and thereby diminish its ability to settle rapidly, as well as failing to entrap the very fine suspended sediment for which the coagulating chemical was added. Provided a very thorough mixing action is obtained in the channels, it is evident that the coagulation basins themselves may be made smaller than they otherwise could be.

A more recent design is found in the Grand Rapids, Mich., plant, where the water after passing through a grit chamber holding about a 26.5-min. supply at a normal rating of 20,000,000 gal. per day, enters a mixing chamber 44 ft. wide by 160 ft. long. The chamber holds 732,000 gal., or about a 53-min. supply at normal rating. The basin is provided with wooden baffles of "around the end type," spaced 3 ft. apart for the full length of the chamber.

Mr. J. W. Armstrong who designed this plant, as well as the plant at New Orleans, states this type of baffle permits the operation of the plant with varying heads of water, and offers reasonably good facilities for cleaning and inspection.

Settlement After Coagulation.—The deposition of coagulated sediment should take place rapidly if the proper amount of chemicals have been added to the water, a thorough mixing of the water with the chemicals has been effected, and temperature conditions are favorable. The same general principles governing the deposition of sediment in plain sedimentation basins, as previously discussed, apply here, except that the suspended particles are

larger and should gravitate toward the bottom more rapidly. A velocity of flow of 1.0 to 1.5 ft. per minute can probably be safely maintained in most cases without carrying too much of the finest material forward to the outlets of the basins. Baffles will probably be found advantageous in most basins of this type. A uniform and even distribution of flow across the entire cross-section of the basin is desirable between the inlets and outlets. Skimming from the surface at the outlets is the proper method for the withdrawal of the settled water. Provision for secondary application of the coagulating chemicals as the water flows through the basins for the purpose of correcting the original treatment, should never be omitted in any well-designed plant. If possible, an auxiliary basin for getting the benefit of this secondary treatment ought to be installed, otherwise too much coagulated material is liable to be carried through the outlets to the filters.

The primary coagulation basins are usually constructed in duplicate in order to be able to clean one of them at a time without interrupting the operation of the whole plant. An adequate number of sumps and drains ought to be provided, so that the deposit when ready to be cleaned out does not have to be moved too far. The size of the drains should be ample to prevent clogging, and plenty of water should be available for flushing purposes.

St. Louis Settling Basins.—The six original settling basins of this water-works plant (Fig. 10) were constructed as plain sedimentation basins. For a number of years since the chemical coagulants sulphate of iron and caustic lime have been used to clarify the water, they have become coagulation basins. They originally consisted of six rectangular masonry tanks 670 by 400 ft. in plan, and placed side by side. The walls are of masonry but the bottoms are of concrete. They were intended to be operated in parallel, but since coagulants have been used the walls dividing adjoining basins have been cut down, so that the water now passes through them in series, entering through four 3 by 3 ft. openings at the end of one of the basins. The water is withdrawn through a masonry conduit at the end of the sixth basin after having passed through the preceding five basins. Sluice gates in each basin, connected with both the inlet and outlet conduits, make it possible to withdraw basins from service for cleaning, and to provide for emergency conditions.

Two additional basins, covering together an area 400 ft. wide by 1,660 ft. long have been constructed of concrete. The two basins are separated by a division wall, and are each 400 by 800 ft. in plan, and have an average depth of about 21 ft. Their combined capacity is 75,000,000 gal. This makes the total capacity of the eight basins 255,000,000 gal. These are the

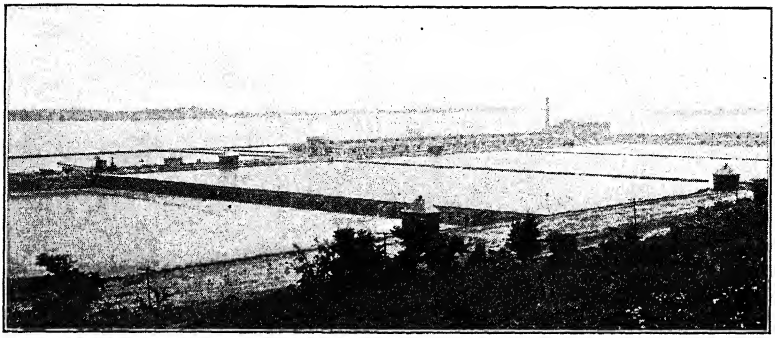


FIG. 10.—St. Louis settling basins.

largest coagulation basins known to the author, and they handle effectively a very turbid water. The effluent from these basins is now filtered.

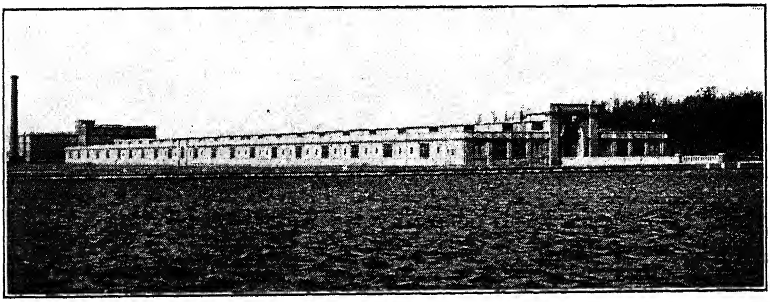


FIG. 10a.—St. Louis filtration plant.

Cincinnati Coagulation Basins.—A good example of coagulation basins which are effective, but which are unprovided with baffles, are the three basins of the Cincinnati plant (Figs. 11 and 12). A lined basin or reservoir 400 by 400 ft. in plan is divided by a concrete wall into two basins, each of which may be operated independently, but which are usually operated together in parallel. The depth of these basins is about 21 ft. The water enters the two

basins through sluice gates at the bottom of a gate house at one end of the division wall. It flows into the bottom of the basins through two reinforced-concrete conduits on either side of the above-mentioned gate house. These conduits are 7 ft. in diame-

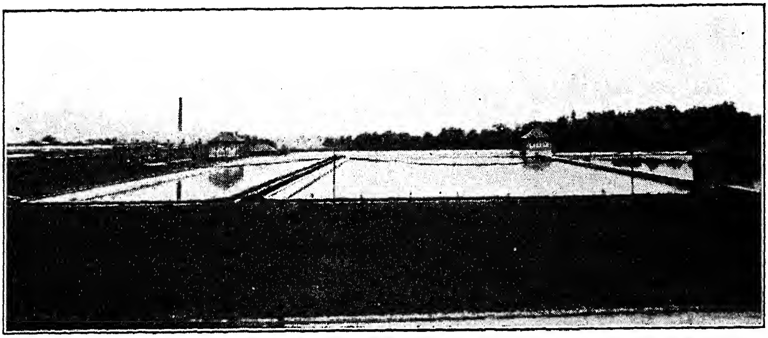


Fig. 11.—Cincinnati coagulation basins.

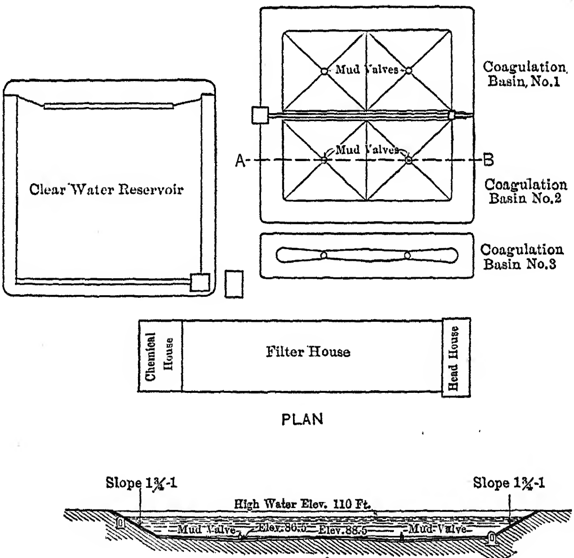


Fig. 12.—Cincinnati coagulation basins, plan and cross section.

ter and have set in the top of each of them twenty-one 20-in. tees. Each of these tees discharges through the two openings in line with each other, and in parallel with the line of flow through the body of the conduit. In this manner the velocity of the counter-

currents of water tend to oppose each other, and to produce a uniform and quiet inflow. The water then passes across to the opposite side of the basins, and is skimmed off at the surface by openings into a steel conduit which conveys the settled water to either a third basin for secondary treatment, or directly to the filters. Two mud valves hydraulically operated are placed in the bottom of each basin. The slope of the bottom toward the sumps of these valves is approximately 2 per cent. The sides have a slope of 1.75 to 1. These basins have a concrete lining covered with hard-burned brick which are grouted in place so as to form a smooth hard surface for flushing out the deposited mud.

The third basin is 400 by 80 ft. and about 16 ft. deep. It is used in series with the two larger basins, and although operated

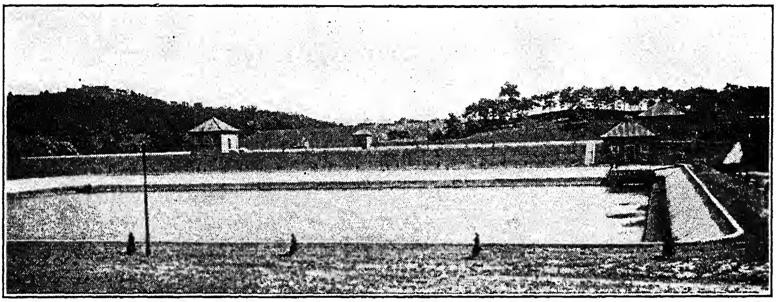


Fig. 12*a*.—Cincinnati filtered water reservoir.

continuously to obtain the maximum period of sedimentation, was originally intended for settling the water after a secondary application of chemicals, when the first application had been found insufficient. Whenever it is necessary to make a second application of chemicals, which is rarely the case, the third basin is used as originally intended.

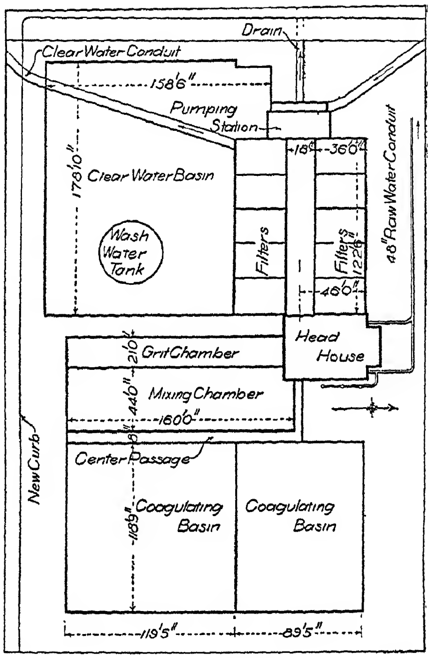
The two large basins have a capacity of 10,000,000 gal. each, and the small basin a capacity of 2,000,000 gal. At the maximum output of this plant a period of 4 to 5 hr. for settlement is possible in these basins.

Grand Rapids Coagulation Basins (Fig. 13).—As an example of well-baffled coagulation basins those at Grand Rapids, Mich.,[1] may be cited. There are two covered coagulation basins in this plant. The smaller basin is 88 ft. 6 in. by 118 ft. 9 in., and holds

[1] "The Municipal Water Purification Plant at Grand Rapids, Mich." *Eng. Record,* vol. 64, p. 379, 1911.

1,134,000 gal. of water, which is about 1 hr. and 22 min. supply at the normal rating of 20,000,000 gal. per day. The larger basin is 118 ft. 6 in. by 118 ft. 9 in., and holds 1,452,000 gal., or about 1 hr. and 44 min. supply at the normal rating. The basins may be operated in series or in parallel.

Mr. J. W. Armstrong in describing this plant notes that in basins having but few baffles, that there is a tendency for the



Plan of Works.

Fig. 13.—Grand Rapids water purification plant, general plan.

water to short-circuit, and for the floc to settle out unevenly in different parts of the reservoir. In order to overcome this difficulty and to maintain a more even distribution of the floc, the baffles in these basins are placed closer than usual, being 15 ft. apart on centers.

CLEANING SETTLING AND COAGULATION BASINS

The method of removing the sediment deposited in settling and coagulation basins should always receive careful consideration in design. Where large amounts of sediment must be removed,

adequate means for handling the accumulated mud are necessary in order not to keep a basin out of service for too long a period. In small plants the time factor is not of so much importance, although even here great inconvenience may arise from limited facilities for handling the accumulated sediment. Basins ought always to be in duplicate, in order that one at a time may be withdrawn from service for cleaning. Where the sediment accumulates rapidly, frequent cleaning appears preferable to allowing a large amount to accumulate, which would require a considerable time to remove.

Concrete or concrete- and brick-lined basins form the best surfaces from which to wash off the mud. Drains should be of ample size, and there should be a good slope toward them. The general method of cleaning employed will depend on the slope of the sides and bottom toward the sumps, and on the distance the mud must be moved. Where the deposit is washed out by streams of water under pressure, the author has found that slopes of less than 1 in 20 greatly retarded the process of cleaning. One and 2 per cent. grades are not enough when cleaning by this method. On such grades the semi-fluid deposit has to be repeatedly pushed forward toward the sumps, thereby losing much time and requiring large volumes of water.

Where scraping is employed to assist the flushing with water, the latter need not be under much pressure, but a considerable volume of water is needed to give a semi-fluid consistency to the mud. At the St. Louis plant horse-drawn scrapers are employed made from 2-in plank 10 in. wide and 12 ft. long. Men follow the teams with hand scrapers, cleaning the bottom as they go and arranging gutters so as to confine the flushing water used to a narrow strip nearest the teams and to prevent its being wasted over the cleaned bottom.

The St. Louis basins have a comparatively flat bottom with a gutter running through the center. This gutter runs lengthwise of the basin and has a 1 per cent. slope to the sewer into which it discharges. No trouble is experienced in moving the mud along the gutter. More difficulty is encountered with the heavy silt and sand which settles near the inlets, and which requires teaming and much water to move.

It has been the experience of the author that sand and silt deposits are easily moved on 8 and 10 per cent. slopes, provided the deposit is first disintegrated by a stream of water under a

pressure of 70 to 75 lb. per square inch. The mud banks in such cases need not be entirely liquefied; but if they are undermined by a stream of water (Figs. 14 and 15) from a nozzle 1 in. or even

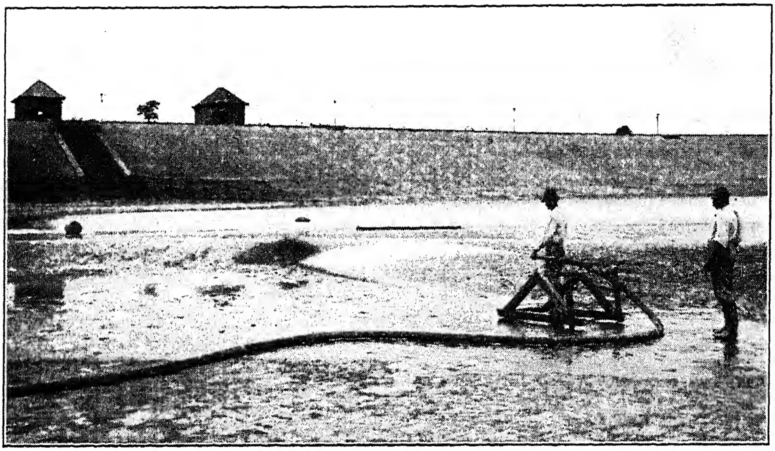


Fig. 14.—Cleaning Cincinnati settling reservoirs.

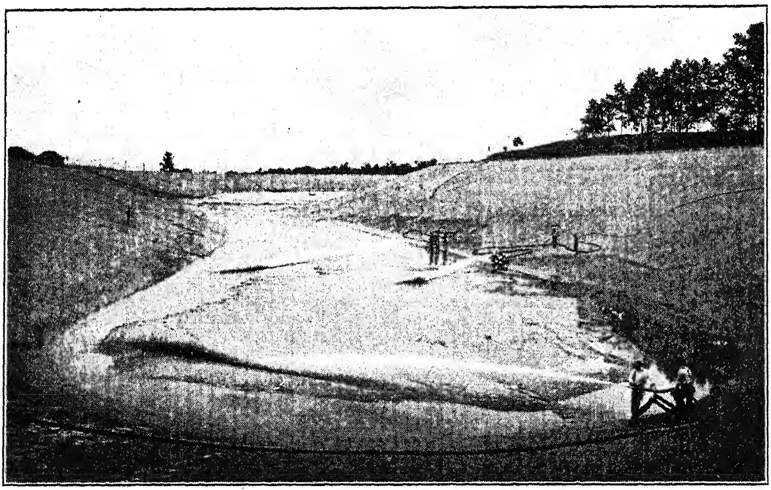


Fig. 15.—Cleaning Cincinnati settling reservoirs.

⅞ in. in diameter and under the above-mentioned pressure, they can be readily floated away to the sump. On the other hand, the gelatinous clay deposits usually have to be well liquefied before they can be moved, although on slopes of 10 or 12 per cent.

the mud can be floated to the sump in larger pieces than when the grades are less. These observations apply particularly to the kind of silt and clay carried by the water of the Ohio River, and to handling large amounts of sediment in large settling reservoirs. Nevertheless, they are believed to be generally true, although the original character of the sediment must necessarily influence the general properties of the deposit. In coagulation basins the applied chemicals or the products of the reactions, such as hydrates of iron, aluminum, magnesium, and carbonate of lime, must of necessity modify the nature of the deposit.

Cost of Cleaning Settling and Coagulation Basins.—The cost of cleaning basins will necessarily vary with the kind and amount of deposit to be removed. This has been found to be the case both at St. Louis and at Cincinnati. At St. Louis the cost seems more dependent on the amount of sand in the deposit than in the amount to be moved. The following table compiled from the records of the Cincinnati Water Department shows the cost of cleaning one of its large plain sedimentation reservoirs.

	Total cost	Estimated cost per cubic yard of sediment removed
Labor . .	$1,100.45	$0.0267
Supplies. . .	440.23	0 0107
Power.	154.44	0.0037
Value of water used in cleaning. .	62.82	0.0015
Value of water wasted in draining .	183.13	0.0044
Total	$1,941.07	$0.0470

N.B. The cost of cleaning per million gallons of water settled was $0.056.

The cost of cleaning settling basins at St. Louis, Mo., in which coagulated sediment is precipitated is shown by the following table compiled from the annual reports of the Water Commissioner.

COST OF CLEANING CHAIN OF ROCKS' BASINS

	1908	1909	1910	1911
Cubic yards sediment removed...	135,108	129,035	182,500	144,200
Total cost. . . .	$2,631.85	$2,947.26	$3,159.11	$2,158.39
Cost per cubic yard ..	$0.0195	$0.0228	$0.0173	$0.0150

The costs given for cleaning the Chain of Rocks' basins are for labor and teams, for furnishing and keeping in repair all tools, boots, etc., used in cleaning, and for moving such apparatus to and from the basins before and after cleaning. It does not include the cost of the water lost by draining or used in cleaning the basins.

In April, 1908, 700 cu. yd. of mud were removed from the Baden storage reservoir at St. Louis, at a cost of $149, or $0.213 per cubic yard. The basins at Bissell's Point were cleaned in August of the same year at a cost of $0.133.

Cost of Cleaning Cincinnati Coagulation Basins in 1911

Labor	$89 67
Power.	24 75
Value of water lost in draining and used in cleaning.	76 52
	$190 94

N.B. Cost per cubic yard of mud estimated to have been removed, $0.024.

For cleaning these basins twice in 1910, when 15,600 cu. yd. of mud were estimated to have been removed, the total cost was found to be $468, or a cost of $0.03 per cubic yard. If the value of the water lost in draining, and that used in cleaning is deducted from this total cost, the cost per cubic yard is practically one-half that given above or $0.015. This latter figure is the one to be compared with those costs obtained in cleaning the Chain of Rocks' basins in St. Louis, since there no charge for the value of the water lost by draining or that used in cleaning is included.

The cost of cleaning reservoirs, passages, and chambers of the Carrollton plant of the New Orleans water purification plant is of interest, on account of the completeness of the detailed list of cost items. The following table is taken from the Report of the Sewerage and Water Board for the year 1911:

Total amount of wet mud removed from reservoirs.	45,000 cu. yd.
Total amount of dry material removed from reservoirs. . . .	18,000 cu. yd.
Total amount of water treated during year.	5,274 M. gal.
Total amount of filtered water required, 4 M. gal., value. . .	$62.40
Total amount of treated water used and wasted, 17 M. gal., value.	174 59
Total amount of raw water used and wasted, 12 M. gal., value.	47.52
Total amount of labor required, value.	302.05

Cost of labor for cleaning per million gallons water treated.	0 053
Cost of water for cleaning per million gallons water treated.	0.049
	$0 102
Total estimated cost of cleaning per cubic yard of dry material removed	0.033

Value of raw water taken at $3.96 per million gallons.
Value of treated water taken at $10.27 per million gallons.
Value of filtered water taken at $15.55 per million gallons.

The cost of cleaning based on the wet mud removed is $0.013 per cubic yard, and is comparable with the figures given for cleaning the Chain of Rocks, basins at St. Louis or the coagulation basins at Cincinnati.

Cost of Clearing Water by Settling.—Mr. S. Bent Russell in a paper[1] discussing the cost of clearing water by sedimentation in reservoirs gives some interesting data on the costs obtained at St. Louis in operating the Bissell's Point basins between 1881 and 1894. These basins were operated as plain sedimentation basins on the intermittent-flow plan. The head lost by this method of operation was 14 ft., and the cost of the increased lift is figured in the following table as one of the six items of cost chargeable against the clarification of the water.

Analysis of Cost of Sedimentation in Bissell's Point Basins at St. Louis, Mo., between 1881 and 1894.

Items	Cost per million gallons settled	Cost per year in per cent. of first cost
1. Interest	$2.820	5 000
2. Depreciation	0.790	1 400
3. Repairs	0 054	0.068
4. Operating	0.281	0.340
5. Cleaning	0 198	0 230
6. Increased lift	0 329	0.585
Total	$4 472	7.623

Mr. Russell concludes that:

"The cost of clearing is dependent upon the quantity of water handled and the proportion of sediment removed. The area or dimensions of the floors will influence the cost. The inclination of floor and drains, etc., are important factors, and the character of the sediment must be considered. This item is of some importance, where there is much

[1] *Eng. Record*, vol. 60, Oct. 16, 1909.

sediment, and to keep the cost within proper limits we are justified in adding considerably to the first cost of the plant."

The cost of plain sedimentation, as shown by the operation costs of the Cincinnati, Ohio, settling reservoirs, was $0.20 per million gallons of water settled in both 1909 and 1910; in 1911 the cost was $0.19, and in 1912 $0.42 per million gallons of water settled. These costs do not include fixed charges, but are the entire cost of operating and maintaining the reservoirs and the grounds about them which are quite extensive. They include in the years 1909 and in 1912, respectively, the cost of cleaning one reservoir. Probably on an average 50 per cent. of the foregoing costs at Cincinnati are chargeable to the upkeep of grounds about the reservoirs, and should not properly be made a part of the cost of sedimentation.

In 1912 labor costs were greater than in 1909, and this together with certain changes made in drainage valves during the cleaning of the reservoir in that year accounts in part for the increased cost. The cost of upkeep of the grounds about the reservoirs in 1912 was over 40 per cent. of the total.

First Cost of Settling Reservoirs.—The first cost of constructing settling reservoirs naturally varies with their size, manner of construction and the general topography of the locality. Reservoirs formed by the damming of a valley and having the land to be flowed stripped of its surface soil, have been found to cost from $135 to $600 per million gallons of capacity. Reservoirs formed by an earth embankment damming a natural ravine, and lined with concrete and brick, like those at Cincinnati, cost nearly $4,000 per million gallons of capacity; while the coagulation basins built principally in embankment and lined with concrete and brick cost nearly $14,000 per million gallons of their holding capacity. On the other hand, masonry-walled basins like those at St. Louis cost from $6,000 to $6,500 per million gallons capacity.

The cost of the coagulation basins of the Toledo, Ohio, water purification plant was $2,500 per million gallons of the daily rated capacity of the plant. The coagulation basins of the Cincinnati plant mentioned above when estimated in a similar manner cost about $2,700 per million gallons of daily capacity. The settling basins of the Columbus, Ohio, purification plant cost $5,630 per million gallons of daily capacity, while the large mixing tanks cost $1,470 per million gallons of daily capacity.

These figures are given merely to show ranges of cost rather than to make any comparisons, since the local topographical conditions, general type of plant and style of construction are so varied that a strict comparison of costs is improper and may be misleading.

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CHAPTER VIII

PRACTICAL EFFICIENCIES OF SETTLING AND COAGULATION BASINS

The practical efficiency of settling basins in the removal of sediment from turbid waters is variable, and depends upon a number of factors which have been discussed in the preceding two chapters. The manner in which the basins are operated, either as a result of unintelligent methods of handling, or because of the demands upon the plant which the operator must meet, are conditions contributing to their inefficiency. How great a percentage of the sediment is removed in the different types of basins is best illustrated by actual examples.

The very finely divided clay sediment in the Allegheny River at Pittsburgh, Pa., is difficult to settle out. The sedimentation basins hold approximately 120,000,000 gal. of water. On the basis of 90,000,000 gal. daily consumption there is a theoretical storage of 32 hr. It has been concluded, however, that the water frequently passes through the basins in as short a time as 11 or 12 hr. The following table compiled from the 1910 report of the Bureau of Water of Pittsburgh indicated a yearly average removal of but 28.2 per cent., and ranged from zero to nearly 49 per cent.

At Cincinnati, Ohio, the sediment in the Ohio River water is possibly somewhat more easily removed by settling. In the settling reservoirs there is provided a considerably longer period of storage, than at Pittsburgh, which probably accounts for the larger percentage of sediment removed.

In the Cincinnati reservoirs there is a theoretical storage for 6.6 days if based on a consumption of 50,000,000 gal. per day and an available storage capacity of 330,000,000 gal. From observations made it has been concluded that the water actually passes through these basins in as short a time as 40 to 48 hr. The relative positions of the inlets and outlets in these reservoirs, as previously described, obviously make complete displacement impossible.

TURBIDITY OF ALLEGHENY RIVER WATER AT ROSS PUMPING STATION AND OF WATER AFTER SETTLING

Averages of 2-hr readings in 1910–1911

Year, 1910	River water			Settled water			
	Average	Maximum	Minimum	Average	Maximum	Minimum	Percentage reduction
February.	38	388	7	22	110	8	42 1
March...	65	366	14	58	200	14	10 8
April	44	300	18	29	110	11	34 1
May .	24	53	10	18	38	8	25 0
June	27	147	13	21	110	10	22.3
July . .	17	27	8	15	25	8	11.7
August . ..	18	29	12	17	23	12	5 5
September . ..	50	158	6	40	140	6	20 0
October ..	19	56	9	20	80	11	.....
November .	35	105	13	29	60	13	17.1
December ...	54	833	8	28	270	7	48 1
1911							
January . .	78	328	16	40	100	16	48.7
Average ..	39	. .	...	28	...	...	28 2

AVERAGE PERCENTAGE REMOVAL OF SEDIMENT BY PLAIN SEDIMENTATION IN CINCINNATI SETTLING RESERVOIRS

Month	Average turbidity				Percentage removal	
	River water		Settled water			
	1911	1912	1911	1912	1911	1912
	Parts per million					
January . .	240	122	85	54	64.6	55.7
February. ...	190	226	105	83	44 7	63 2
March.......	140	360	62	190	55 7	47.5
April.........	160	291	57	105	64.4	64.0
May..........	55	202	18	97	67.3	52.1
June..........	76	100	24	20	68.4	79.9
July...........	50	712	24	190	52 0	73.4
August.	64	385	27	170	57.8	55.8
September . .	410	328	119	140	71 0	57.3
October.	257	60	102	13	60 3	78.3
November. ..	148	76	40	16	73.0	79 0
December.. .	128	87	36	25	71.9	71.3
Average ..	159	245	58	92	63.5	62.5

At the New Orleans purification plant the turbid Mississippi River water is pumped directly into so-called grit reservoirs, and flows from the latter into mixing channels, where the water receives the lime and such small amounts of coagulant as are used at this plant. It then flows to the settling basins, and from thence to the filters. The following table taken from the 1911 report of the Sewerage and Water Board of New Orleans indicates the reduction in turbidity effected at various stages of the process.

TURBIDITIES OF MISSISSIPPI RIVER WATER AND EFFLUENTS OF GRIT RESERVOIRS AND COAGULATION RESERVOIRS AT NEW ORLEANS PURIFICATION PLANT

	River water			Effluent grit res.			Effluent coag res		
	1909	1910	1911	1909	1910	1911	1909	1910	1911
	Parts per million								
Maximum	1,600	1,700	1,400	1,550	1,450	1,250	340	525	280
Minimum	80	55	150	75	55	130	1	2	5
Average	525	550	500	475	450	425	44	32	32

It will be noted that the reduction in the turbidity of the river water after passing through the grit reservoirs ranges from 8 to 18 per cent. on an average, while the reduction after passage through the coagulation reservoirs averages over 90 per cent. In all probability the grit reservoirs really remove considerably more sediment by actual weight than is indicated by the turbidity readings. They are to be regarded as settling chambers for the removal of the coarse silt and sand only, and not for depositing the finer clay particles.

The purification plant at St. Louis, Mo., offers some interesting data on the removal of sediment by coagulation with lime and sulphate of iron, and the effect of the series of settling basins through which the treated water is passed. Mr. W. F. Monfort, chemist at this plant, states in the report of the Water Commissioner issued in January, 1913, that:

"When a water properly treated is passed through the basins 97 to 99 per cent. of the suspended matter is precipitated in the first basin, the percentage removal depending upon the character and quantity of solids contained, the temperature, wind velocity, and direction, and the amount of sludge in the filling basin. In passing succeeding basins the remaining suspended matter undergoes a further reduction to one-

half or one-sixth, likewise dependent upon velocity, size of particles, wind, and temperature. The major portion of clarification is, however, accomplished in the filling basin. There is no provision for applying chemicals after water enters the basins. Efficiency of the plant, therefore, depends primarily upon the volume of water which can be satisfactorily clarified in the filling basin (Fig. 16).

"The weight of suspended matter in the effluent from successive basins varies with the weight of solids carried by the raw water. Some illustrations are given:

	Parts per million	Per cent removal	Parts per million	Per cent removal	Parts per million	Per cent removal	Parts per million	Per cent. removal
River	1,444	..	3,000		4,500	.	1,000	
Basin 1	14.0	99.01	47	98.5	95	98.0	25	99.75
Basin 2	12.1	99.16	21	99.3	50	98.8	20	99.80
Basin 3	8.4	99.40	14	99.5	35	99.2	10	99.90
Basin 4	7.1	99.50	12	99.6	20	99.5	10	99.90
Basin 5	5.8	99.60	10	99.7	15	99.7	5	99.95
Basin 6	5.6	99.60	10	99.7	15	99.7	5	99.95
Basin 7		.	.		.		5	99.95
Basin 8.	.	...	.		.		5	99.95

"A change of pumping from a rate of 60,000,000 gal. per day to 90,000,000 gal. has increased the suspended matter in the treated water from 2 to 7 parts per million, and a further sudden increase to 120,000,000 gal. per day has caused a further rise of 10 to 13 parts.

"In the spring when the temperature of the water in the river and basins is rising the sludge is less subject to disturbance than when, as in the fall and early winter, with falling temperature, the influent water, being more dense than the warmer water in the basins, passes downward over the sludge, causing it to carry over the weir from the filling basin.

"In the fall with lowering atmospheric temperatures, the sludge and water in the bottom of the basin are sometimes 1°F. or more warmer than the surface water of the basins and river. Circulation in the basins, therefore, is effective in changing the course of the influent water currents, making them deeper, and increasing the scour.

"The sludge is further subject to disturbance by wave action when high winds prevail for a day or two, as frequently occurs in March. In such cases the amount of suspended matter carrying over the weir from the filling basin shows a marked increase.

"The basins at the Chain of Rocks (52 acres) are all uncovered, all used in series, and, therefore, subject to disturbance by each of the agencies affecting their successful working."

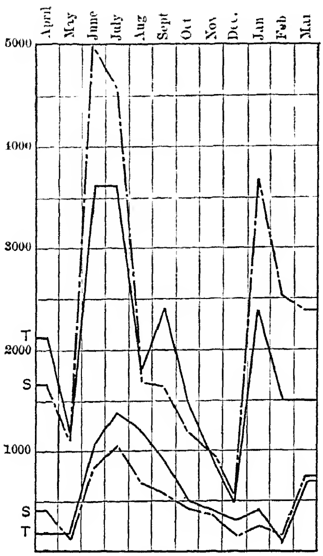

Diagram No 1
MAXIMA AND MINIMA OF TURBIDITY AND SUSPENDED MATTER, PARTS PER MILLIONS, IN RAW WATER

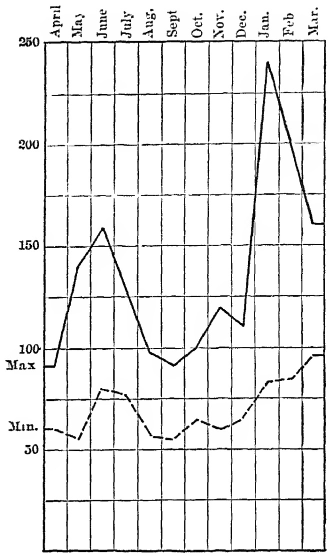

Diagram No. 2
COEFFICIENT FINENESS OF SOLIDS IN RAW WATER

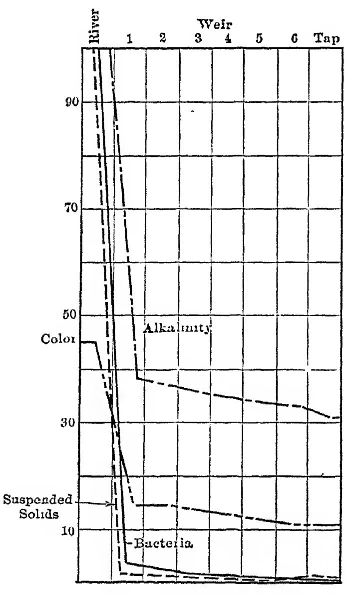

Diagram No. 5
REDUCTION THROUGH THE BASINS

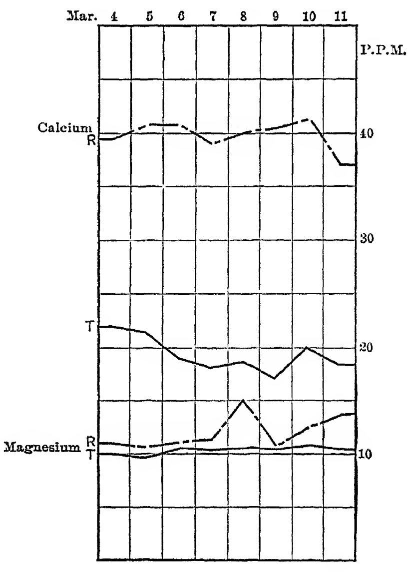

Diagram No. 7
COMPARISON OF CALCIUM AND MAGNESIUM IN RAW AND TREATED WATER.

FIG. 16.—Diagrams showing characteristics

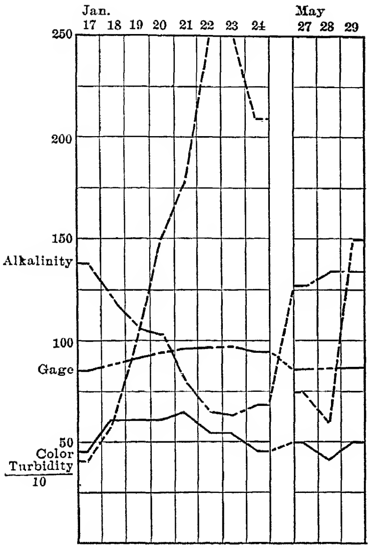

Diagram No. 4
TYPICAL FLUCTATIONS IN CHARACTER OF RAW WATER

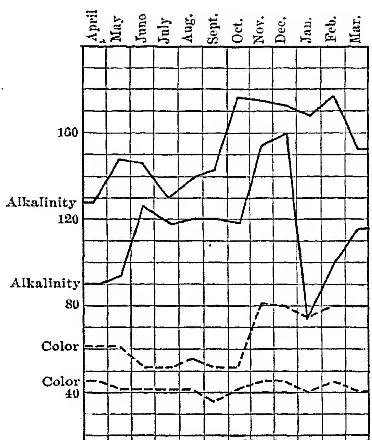

Diagram No. 3
MAXIMA AND MINIMA OF ALKALINITY AND COLOR IN RAW WATER

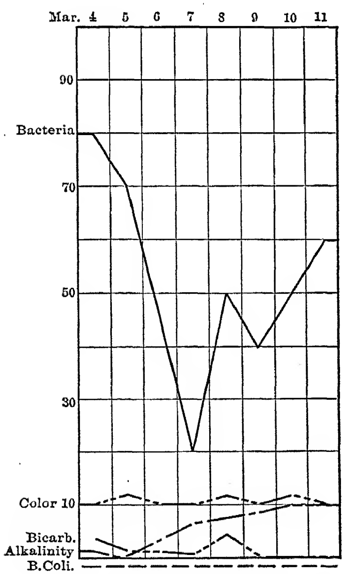

Diagram No. 8
CHARACTERISTICS OF TREATED WATER

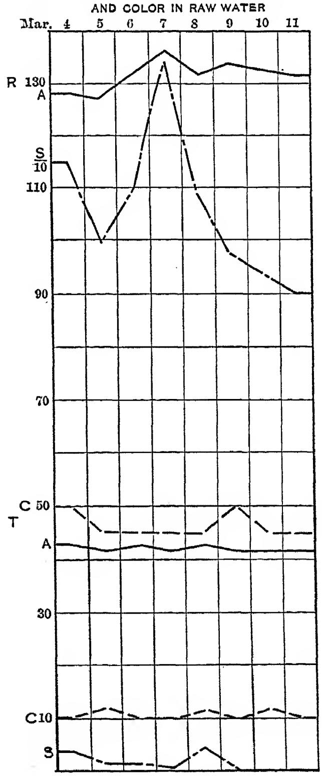

Diagram No. 6
COMPARISON OF ALKALINITY, SUSPENDED SOLIDS AND COLOR IN RAW AND TREATED WATER

of raw and treated water at St. Louis, Mo.

The author's experience with the settling and coagulation basins of the Cincinnati purification plant confirms the observations of Mr. Monfort on the effect in change in rate of flow, temperature, velocity of wind and amount of sludge in the reservoirs and basins.

Summaries of the percentage removal of sediment by the process of plain sedimentation followed by coagulation and filtration at the Cincinnati plant are given in the following table:

	Turbidity, parts per million				
	1908	1909	1910	1911	1912
River water . . .	190	225	190	159	245
Settled water	100	85	80	58	92
Coagulated water	32	23	21	15	16
	Percentage removal				
By settling reservoirs.	48.4	62.2	57.9	63.5	62.5
By coagulation basins.	34.6	26.7	31.1	27.0	31 0
By filters	17 0	10 2	11 0	9 5	6 5

Bacteria in River, Settled, Coagulated and Filtered Water

	1908	1909	1910	1911	1912
	Bacteria per cubic centimeter				
River water..	7,000	9,300	8,900	13,790	11,130
Settled water. .	3,400	2,500	3,200	3,140	3,945
Coagulated water.	700	475	750	776	898
Filtered water	100	75	75	39	26
Average percentage removal...	98 5	99.2	99.2	99.7	99.8

The effect of coagulation and settlement upon suspended sediment and the bacteria in treating the Scioto River water at the Columbus, Ohio, water-purification plant is shown in the following tables. In this plant lime is used to soften the water and only small amounts of chemical coagulants are applied.

Month	Turbidity, parts per million					
	1909		1910		1911	
	River water	Settled water	River water	Settled water	River water	Settled water
January	7		94	21.0	137	7.0
February	256	...	77	12.0	146	8.0
March	109	17.0	66	12.0	28	1.0
April	102	21.0	22	4.0	117	5.9
May	196	28.0	41	8.0	61	3.8
June	92	14.0	15	3.0	19	1.5
July	86	9.6	24	4.0	14	0.6
August	24	4.0	19	4.0	18	0.0
September	20	4.0	16	1.3	39	0.1
October	13	2.0	44	3.6	72	3.4
November	17	1.4	6	1.6	98	5.0
December	107	13.0	20	1.0	64	2.5

Month	Bacteria per cubic centimeter					
	1909		1910		1911	
	River water	Settled water	River water	Settled water	River water	Settled water
January	435	12	52,000	1,600	36,000	43
February	32,825	2,146	15,500	190	8,000	9
March	18,228	828	20,000	190	1,500	7
April	8,500	533	2,600	28	12,400	27
May	16,309	3,523	3,300	24	3,546	383
June	3,952	180	1,100	12	800	9
July	4,009	42	1,900	7	1,700	40
August	1,888	83	550	4	1,800	190
September	1,171	65	700	2	2,400	800
October	707	100	2,200	42	6,500	160
November	1,300	60	300	10	44,000	275
December	21,500	65	12,000	14	17,000	120
Averages	9,235	636	9,338	177	11,470	172

The bacterial reduction produced by settling the Susquehanna River water with sulphate of alumina is shown in the following table of averages:

Bacteria per Cubic Centimeter in River Water and Settled Water and Percentage Reduction Effected

Year	Bacteria per cubic centimeter		Percentage reduction
	River water	Settled water	
1906	12,372	3,228	73.91
1907	10,712	4,504	57.96
1908	4,949	1,662	66.43
Average	9,344	3,131	66 49

It is obvious that the amount of coagulant used in treating the water, which contains suspended sediment, will have a marked influence on the apparent efficiency of settling basins. The more coagulant used the greater the precipitation of both the clay particles and the bacteria.

References

1. Report of Sewerage and Water Board of New Orleans, La., for 1911.
2. Report of Bureau of Water of Pittsburgh, Pa., for 1910.
3. Report of Water Department of Cincinnati, Ohio, for 1911–1912.
4. Report of Division of Water of the City of Columbus, Ohio, for 1911.
5. Report of Board of Commissioners of Water and Lighting Department, Harrisburg, Pa., for 1908.
6. W. F. Monfort: "The Water Supply of St. Louis, Mo." *Reprint* from Report of Water Commissioner for 1913.

CHAPTER IX

FILTRATION OF WATER

Neither plain sedimentation nor that assisted by the use of coagulating chemicals, or both processes combined can, as a rule, produce complete clarification of turbid waters. Excellent results may be obtained, however, provided the periods of sedimentation are sufficiently prolonged and enough coagulating chemicals are used. As a method of treatment preceding filtration, however, they are very effective, since they naturally reduce the work to be done by the filters. The efficiency of settling as a method for the removal of the bacteria has been illustrated by practical examples in the preceding chapter. In this chapter the part played by filtration in the purification of water will be considered.

THEORIES OF FILTRATION

Various kinds of material may be used for filtering water, but sand and gravel are the most practical materials for this purpose and are almost universally employed. The passage of a natural water through a layer of sand and gravel of the proper character and thickness, has the effect of removing material in suspension in the water. The effluent from the filter in usually clear, having been freed from visible suspended particles, and is also found to have had removed from it a large proportion of the bacteria. Even vegetable coloring matter and colorless organic matter, which are probably in suspension in a colloidal form, are to a certain extent removed from water containing them by filtering through sand.

The several factors directly affecting the efficiency of sand filters are the thickness of the bed, the size and uniformity of the sand grains, the amount of organic matter in the water and adhering to the sand grains, the kind and amount of coagulating compounds which may have been employed, the temperature of the water, and the rate of flow. Why these factors influence the process of filtration requires a more extended discussion of the probable structure and action of a filter when in operation.

Function of a Filter.—The primary function of a filter is that of mechanically straining out suspended particles. These particles may and usually do vary greatly in size. It can be readily understood how particles larger in size than the interstices of the sand bed will be strained out. It is not so easy to understand how particles with diameters of perhaps 0.0001 mm., and even much smaller are removed. The submicroscopic particles are much smaller than the bacteria, and yet a filter is able to strain them out of the water in which they are suspended. Evidently agencies are operating within the filter bed which are remarkably effective in holding back these fine particles, and which do not depend alone upon the sand grains.

For a long time it has been known that a sand filter was more effective after it had been in service awhile than when first started. Rapid sand filters as well as slow sand filters appear to undergo this improvement. The more organic matter contained in the water being filtered, the quicker this so-called "ripening" is effected. Apparently it is the organic matter which plays the chief part in the straining process. This organic matter may be either living or dead, that is, it may be organized in the forms of plant life such as the bacteria, algæ, diatoms and lower forms of animal life, or it may be unorganized like the vegetable coloring matter and the colorless nitrogenous and carbonaceous compounds characteristic of natural waters.

Real Structure of a Filter.—A close examination of a well-"ripened" sand filter reveals the sand grains covered with a very thin film of a gelatinous character, which is most abundant near the surface of the bed, but appears to permeate it to some depth. This material is evidently in part composed of organic matter both living and dead. It is in the colloidal form and seems to fill partially or wholly the interstices of the bed, and to be supported by the sand grains. Not all of this material between the sand particles is of an organic character. Some of it is inorganic, but it too is apparently in the colloidal form.

It will be easier now to conceive of the sand bed as a framework or skeleton of sand grains supporting and carrying the delicate structure of the filter proper, which latter consists of the colloidal matter deposited and developed within the bed. In a rapid sand filter, which is frequently disturbed by washing, the colloidal matter has less chance to develop and less opportunity for lodgement in the openings between the sand grains than in a

slow sand filter. Nevertheless, when organic matter is plentiful in the water being treated, the sand grains become coated, and not even violent washing of the bed will entirely remove it. Moreover, in this class of filters colloids produced by the reaction of certain chemical compounds which are added to the water, are employed to assist the natural colloids in their straining action.

Properties of the Colloidal Form of Matter with Reference to Filtration.—The filtering action of a sand bed must, therefore, be attributed to the properties of the colloids deposited or formed therein. A consideration of some of the properties of this form of matter will perhaps explain the ability of filters to perform the work which is so characteristic of them. The colloidal state of matter can be considered as a mesh structure, or a sort of porous framework. Through it water diffuses readily, but suspended colloids contained in the water will be held back. Thus the bacteria, which are of a colloidal nature, are unable to pass through the mesh-like structure of the colloids attached to the sand grains, or are held back by adhering to the surface of the colloidal matter upon the sand and in the passages between the sand particles. Inorganic suspended colloids are likewise prevented from passing through the jelly-like colloids within the bed, either because they have been previously coagulated, whereby the particles have become too large to pass through the openings, or because they stick to the surfaces of the colloidal matter within the bed, the same as do the bacteria and other matter too large to pass through the capillary passages.

The adhesion of colloidal particles to each other may be and probably is more than a mechanical attachment, that is, it may be an electrical attraction. It is known that coagulation of suspended colloidal particles is effected by the ions of an electrolyte having an electric charge of an opposite sign from that of the colloid. By neutralization of the electric charges of opposite signs coagulation is effected. It is also known that absorption of ions occurs on the surfaces of colloid particles, and it may be that coagulation by electrolytes is a phenomenon of adsorption. It thus becomes conceivable that within the mesh-like structure of the colloidal matter permeating the sand bed minute coagulation and sedimentation chambers exist in which the suspended colloidal matter in the water being applied to the bed is deposited and held. If such an explanation is accepted, then there must come a time in the operation of any filter when it can no longer

remove matter in suspension in the applied water, *i.e.*, these coagulation and sedimentation chambers and passages become overloaded. When this occurs the filter becomes ineffective. This is a well-known condition in filter operation.

A sand filter, through which a colloidal solution of ferric hydroxide is passed, retains the hydroxide and delivers a clear effluent for a certain length of time. Only a quite definite quantity is thus held back, after which the ferric hydroxide passes through unchanged. "It is reasonable to assume that the positive (electrically charged) colloidal particles (of ferric hydroxide) are discharged and retained by the negatively charged sand grains."[1] This explanation seems reasonable, and probably very similar conditions are possible within a rapid sand filter when it is filtering water treated with a chemical coagulant. Overloading or inability of a filter to pass clear water through it, after it has become clogged with sediment, is characteristic of both slow and rapid sand filters.

Explanation of Some Well-known Filtration Phenomena.—In the experiments on filtration undertaken at Louisville, Ky., in 1896-97, it was shown that by the addition of sulphate of alumina to the turbid Ohio River water a theoretical reduction in the alkalinity did not result, and that the kind and amount of the suspended clay present materially affected the proportion of the carbonate alkalinity used up.

Suspended matter,[2] parts per million	Percentage which the actual reductions in alkalinity were of the theoretical
200	85
400	80
800	75
1,200	65

The influence upon the amount of sulphate of alumina unaccounted for by reaction with the alkaline carbonates in the water, when the amount of suspended matter by weight was practically the same, is shown by the following table taken from this same report.

[1] EMIL HATSHEK: "Introduction to the Physics and Chemistry of Colloids."

[2] GEORGE W. FULLER: "Water Purification at Louisville."

Sample Nos	Suspended solids, parts per million	Percentage which the actual reduction in alkalinity was of the theoretical
1.	500	57
2.	534	74
3.	584	77
4.	516	80
5.	558	84
Average.	550	74

Mr. George W. Fuller in this work also found "that with successive equal amounts of applied sulphate of alumina the work accomplished is not regularly progressive, and that for some reason in the ordinary river (Ohio) water the specific efficiency of the first portion of chemicals is very low, and less than that of subsequent ones."

All of these facts find an explanation in our present knowledge of the properties of colloids. Loss of chemical coagulant is due to adsorption upon the surfaces of the colloidal particles. The ineffectiveness of the first portions of chemicals added is probably due to the fact that a definite minimum concentration of the electrolyte in every case is necessary to produce coagulation. This feature of coagulation has been very extensively investigated for a large number of electrolytes, and the minimum amounts necessary to produce coagulation have been accurately determined.

For example, in a colloidal solution of arsenious sulphide the minimum concentration of a large number of electrolytes, which would bring about coagulation of this colloid, has been determined. The striking feature of the results obtained was that the amount of the salts containing monovalent cations required to produce coagulation was roughly 70 or 80 times that needed of salts containing divalent cations, and 600 times that necessary to be used in the case of salts with trivalent cations. It is, therefore, evident why the salts with trivalent elements like aluminum and iron are the best chemicals to be used in coagulating the suspended colloidal matter in natural waters, since the weight of the electrolyte needed is relatively small.

In water softening the precipitates formed are of a colloidal character, and assist materially in clarifying the water, especially if other suspended material is present. Certain of the precipi-

tates, however, are very slowly deposited, as they appear to be held in colloidal solution for long periods. The so-called "after-deposits" of softened waters may be ascribed to this form of matter.

The effectiveness of "roughing filters" must be due very largely to the action of the colloids formed and deposited within the filter bed. The relatively large size of the gravel and coarse sand of which such filters are usually constructed, as compared with the fine sand used in slow and rapid sand filters, appears to confirm the theory that the real filtering medium of any filter is the colloidal matter within the filter bed.

If a sand filter bed is composed of fairly uniform sized grains, or if those of like size are stratified, as is the case in those rapid sand filters in which wash water is applied at high velocity, it is evident that a more even distribution of the colloidal matter in the bed is produced, and a more uniform rate of flow through all parts of the bed is possible. Too great a depth of bed imposes a needless amount of friction to the flow of water through sand filters, although too shallow a bed may not produce enough resistance with a consequent tendency to uneven rates of filtration.

The temperature of the water necessarily affects its viscosity. The more viscous it is, which is the case at low temperatures, the more frictional resistance it offers to passage through the bed. A low temperature of the water also appears to affect adversely the colloidal content of the bed, since the efficiency of sand filters is always lower during the winter months.

A possible explanation for this fact may be that during the warmer months of the year, when the temperature of natural surface waters is the highest, the microscopic plant and animal life is most abundant. In other words, the quantity of colloidal matter brought to the sand bed is much greater when the water is warmer, and renders the filter relatively more efficient than when the water is at its lowest temperature. If high turbidities due to suspended clay particles occur during this period of low temperature, the ratio of this class of colloids to the organic jelly-like colloidal matter of the filter is much greater. Hence, it may be that a filter under such circumstances becomes less efficient, because of an actual diminution in the quantity of the real filtering medium within the sand bed. In actual practice in operating rapid sand filters such difficulties are overcome by

increasing the amount of coagulating chemicals, *i.e.*, by providing more colloidal matter in the form of aluminum or ferric hydroxide.

The author has personally noted in operating rapid sand filters, when a sudden increase in the clay particles producing turbidity took place in the warm months of the year, that the filters were able to successfully clarify a water with four and five times the turbidity that they did during the winter months, and that a relatively small increase in the quantity of applied chemical coagulants sufficed to maintain a clear effluent. If the greater content of organic colloids in the bed at such a time was not the cause for the better efficiency of the filter, it is rather difficult to conceive of an explanation.

The rate of flow of water through a filter must be properly regulated if a well-clarified and purified effluent is desired; but the rate can be varied widely provided a change from a lower to a higher rate is not made too rapidly, and certain maximum limits for a given depth and kind of bed are not exceeded. Uniformity of flow through all parts of the sand bed is absolutely essential for obtaining good results, and it is easy to see why this is the case if the colloids are regarded as the real filtering medium. Too rapid a flow must have the effect of washing out or breaking down the colloidal structure, and of subjecting it to rapid overloading with suspended material in the water applied to the filter.

Enough illustrations of the phenomena associated with filter operation have been given to show that we must look to the physics and chemistry of the colloidal form in which both organic and inorganic matter may exist to explain in a rational manner the facts of the filtration of water. Biology undoubtedly plays an important part in producing and developing colloidal matter within the filter; and chemical changes which are incidental to the activity of the life and death of the organisms, are a part of the derived phenomena. However, the principles of filtration are in reality based upon the physical properties of the colloids, and upon them must be founded any rational theory of filtration.

References

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2. Paul Rohland: "The Colloidal and Crystalloidal State of Matter."

3. A. A. Noyes: *Jour. Am. Chem. Soc.*, vol. 27, No. 2.
4. Whitney and Blake: *Jour. Am. Chem. Soc.*, October, 1904.
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6. Ellms and Snell: "Colloidal Suspensions and Their Relation to Problems in Water Purification." *Proc.* Am. Chem. Soc., June 22, 1905.
7. George W. Fuller: "Water Purification at Louisville, Ky."

CHAPTER X

PRELIMINARY TREATMENT OF WATER FOR SLOW SAND FILTERS

Many waters require but little clarification in order to make them acceptable for domestic purposes, although from a sanitary standpoint constant purification by means of filtration is essential. For this class of waters filtration through sand beds of large area at a comparatively low rate of flow may be practised for the larger part of the year with excellent results. However, at certain periods, such as follow a heavy rainfall and subsequent flooding of the streams, these waters may become quite turbid with finely divided suspended matter. Slow sand filters will not continuously clarify such waters, and in consequence preparatory treatment has become quite general.

This preliminary treatment may consist of the coagulation of the suspended matter with chemicals and settling out as much of it as possible in reservoirs before passing the water to the filters. In so far as this method of procedure is followed, it differs in no way from that described under coagulation and sedimentation in a preceding chapter.

The efficiency of this type of preliminary treatment for the preparation of turbid waters for filtration through slow sand filters, is strikingly shown in the case of the Washington, D. C. filter plant. In 1908, Mr. Francis F. Longley, assistant superintendent of the plant, conducted[1] a series of experiments on Potomac River water for the purpose of studying the effect of varying periods of sedimentation (Figs. 17 and 18) with and without coagulants, and of the subsequent passage of the water through various types of preliminary filters, followed by filtration through slow sand filters.

His experiments showed, that with one day of sedimentation followed by filtration through either one of two different types of pre-filters, and then through a slow sand filter, that the

[1] Francis F. Longley: "Experiments on Preliminary Treatment of Potomac River Water at Washington, D. C." *Eng. Record*, vol. 57, June 27, 1908.

process failed to remove all of the turbidity. Even after 3 or 4 days of sedimentation followed by triple filtration through slow sand filters, a perfectly clear effluent could not be obtained.

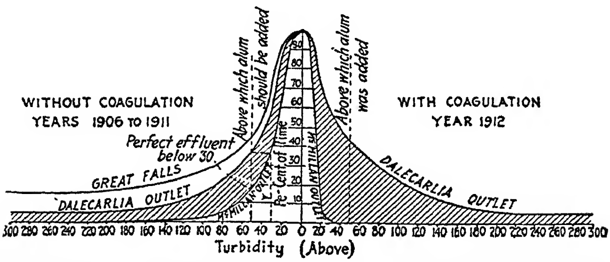


FIG. 17.—Diagram of Potomac river water turbidities and effect of coagulation with alum.

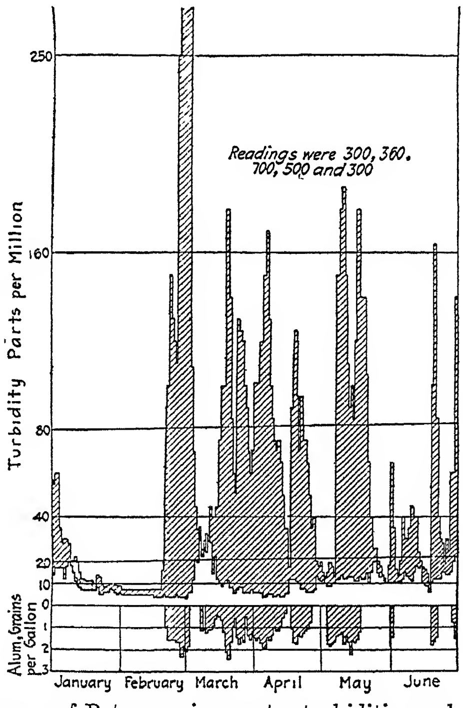


FIG. 18.—Diagram of Potomac river water turbidities and effect of coagulation with alum.

Mr. Longley, therefore, concluded that the complete removal of turbidity

"can not be effected by any device applicable to large quantities of water, that involves purely mechanical principles in its operations."

The clarification of the water was effected, however, by the use of sulphate of alumina, which produced coagulation of the minute clay particles in suspension. The coagulated water was next settled in sedimentation basins and then filtered through a slow sand filter. He, therefore, concluded that:

"the desired improvement in the water (supply of Washington) can be effected by occasional coagulation with subsequent thorough sedimentation in the two existing reservoirs. This process is so entirely flexible that with its use the final product of the filters so much desired, is assured."

The soundness of Mr. Longley's deductions were fully verified 4 years later by results obtained in treating the water supply of Washington as he had outlined. These results are described by Mr. W. F. Wells in a recent paper,[1] and are summarized below.

"1. Seven days' storage is not sufficient to prepare the Potomac water, when the turbidity is above 100, for satisfactory slow sand filtration.

"2. Preliminary treatment with alum introduces an ideal flexibility into the system, whereby the turbidity of water flowing on to the filters may be kept uniformly below 20 (or lower if desired).

"3. Filtration of this water yields a water of constant purity, perfectly clear, and with less than 10 bacteria per cubic centimeter.

"4. The few bacteria surviving filtration are mostly harmless hardy soil forms, making it inadvisable to add hypochlorite of lime for sterilizing purposes.

"5. Cutting down the peak load greatly reduces the cost of the filter operation, and the treated water keeps the beds in better condition.

"6. The rate of filtration following alum treatment can be more than doubled to advantage, and an economical balance struck between the rate of filtration and the quantity of alum."

Pittsburgh's Contact Roughing Filters and A-frame Baffles.— An interesting method for the preliminary treatment of the Allegheny River water prior to its passage through slow sand filters at Pittsburgh, Pa., has been worked out by Messrs. Johnson, Weston and Drake.[2] The necessity for increasing

[1] Wm. F. Wells: "Some Notes on the Use of Alum in Connection with Slow Sand Filtration at Washington, D. C." *Proc.* Am. Water-works Assn., June, 1913.

[2] G. A. Johnson: "Preliminary Treatment of Water for Slow Sand Filters at Pittsburgh, Pa." *Eng. News*, vol. 68, Oct. 3, 1912.

the yield of the slow sand filters of this large plant arose because of the peculiar character of the water which it is obliged to handle. This water is a mixture of the flow of the Allegheny and Kiskiminetas Rivers. The water of the former river is polluted by manufacturing wastes from oil refineries, tanneries and distilleries. In consequence it contains large quantities of organic matter and considerable coloring matter. The Kiskiminetas River water is fouled by the acid wastes of a coal mining region. These two streams unite about 15 miles above the filtration

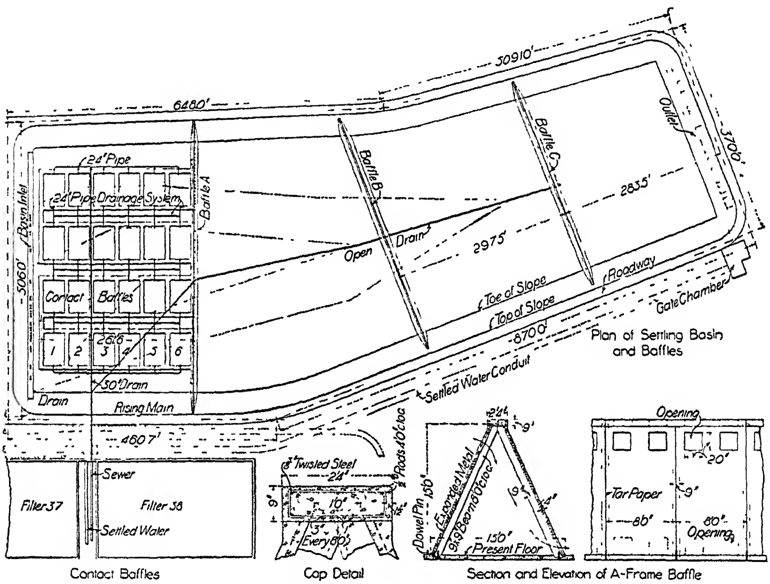


FIG. 19.—Plan of contact baffles and details of A-frame baffles at Pittsburgh filtration plant.

plant, and a reaction between the acid and alkaline waters of the two streams results, which causes coagulation and the production of much matter in the colloidal form. This colloidal matter is formed partly because of the irregular mixing and varying proportions of the waters of the two streams containing these different kinds of suspended and dissolved matters. The petroleum oil wastes further complicate conditions especially in cold weather, when the paraffin of the oils deposit in the sand bed and clog it rapidly. The yield of these filters between scrapings has been cut down at times as much as 70 or 80 per cent. on account of these adverse conditions.

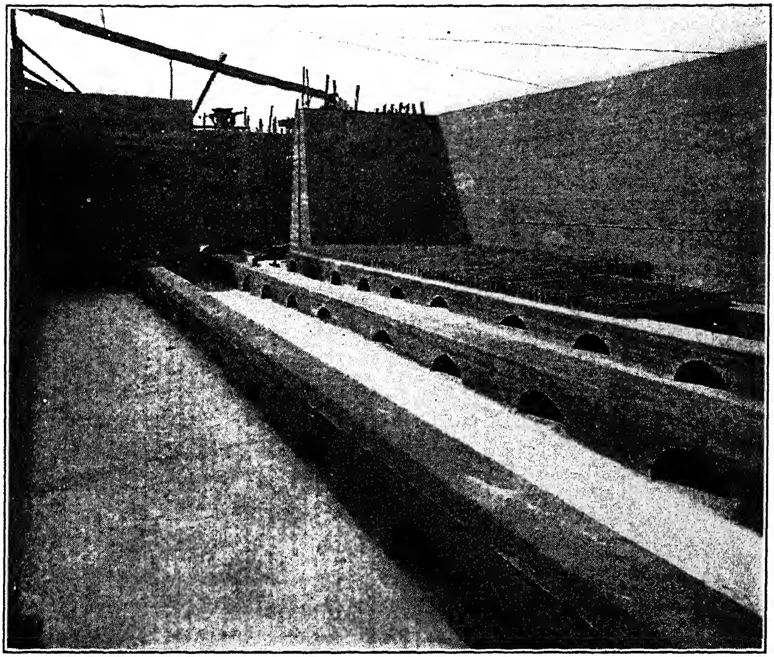
In the experimental investigation which preceded the design and construction of the preliminary treating plant, it was found that the addition of lime to neutralize the acid condition of the water availed little toward preventing the sand beds becoming clogged and caked. It was also found that the use of bleaching powder assisted very materially in lengthening the periods of service of the filters, but for what reason it was not clearly understood.

The plan devised to remedy these difficulties consisted in fitting one of the two 54,000,000-gal. concrete sedimentation basins (Fig. 19) with 24 contact baffles or roughing filters of very coarse stone, and with two hollow A-frame baffles. The second basin was also to be fitted up later on. The object sought in these devices was less the removal of suspended colloidal matter by actual straining, than it was to effect such a mixing and reacting of the various constituents of the water as would permit of their subsequent settling out in the sedimentation basins. These baffles would also be serviceable when coagulants were applied to the water by hastening the chemical reactions.

The contact baffles (Fig. 20) or roughing filters were arranged in four batteries of six units each in one end of the large sedimentation basin. Each unit was a concrete box 61.5 by 41.6 ft. inside dimensions, and about 17 ft. deep. They were each provided with a false bottom composed of four rows of concrete longitudinal beams of arched construction resting on the floor of the tank, and supporting a series of 3 by 8-in. reinforced-concrete beams 7 ft. 4 in. in the clear, and spaced 2 in. apart. On this false bottom 8 ft. of gravel and coarse stone were placed, ranging in size from 3 in. in the bottom layer to ½ in. on top.

The water is admitted to each contact baffle through a 24-in. inlet pipe with its invert at the level of the surface of the gravel filling. Under the usual conditions about 4 ft. of water stands on top of the gravel. A 20-in. pipe at the end of the tank opposite the inlet serves as the outlet or effluent pipe. The cleaning of the contact filters is effected by rapid draining through the coarse gravel and stone to appropriate drain piping by means of quick-opening valves. The roughing filters are operated at the rate of 75,000,000 gal. per acre daily.

From the 24 contact or roughing filters the water passes through openings in a concrete wall to the main body of the sedimentation basin. In this part of the basin are provided two



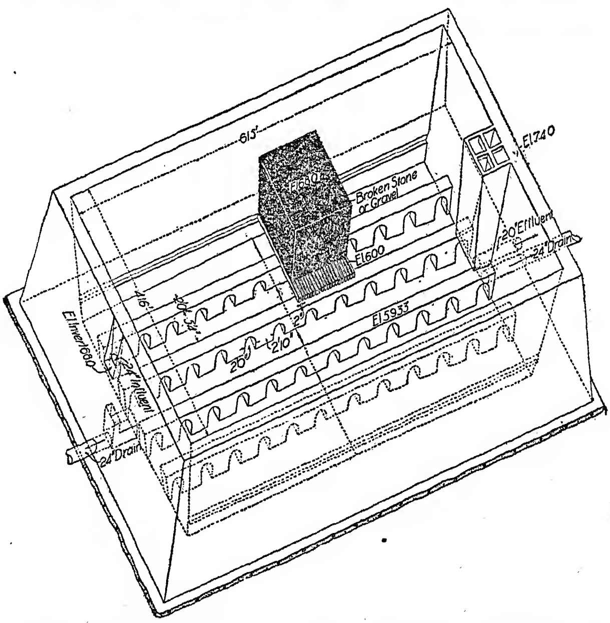


Fig. 20.—Isometric view of contact baffle unit, Pittsburgh filtration plant.

A-frame baffles extending the full width of the basin and keyed into the walls of the latter on either side. These A-frames are hollow and are 15 ft. in height. The water enters near the top of the A-frame through openings provided, and after passing down inside the baffle flows out at the bottom into the next section of the sedimentation basin. A sludge drain extends along the bottom of the basin and through the baffles to be used in cleaning out the deposited sediment.

PRELIMINARY RAPID SAND FILTERS

To avoid the use of coagulating chemicals preliminary filters have been designed and installed in different plants for the purpose of removing a portion of the suspended matter, so that the slow sand filters may not become too quickly overloaded. These filters are designed with the idea of mechanically straining out as rapidly as possible the coarser particles, and are, therefore, constructed of rather coarse gravel and sand, and are operated at relatively high rates of flow, viz., from 28,000,000 to 100,000,000 gal. per acre per day. The effluent from these filters is passed through slow sand filters for a second and final filtration. A water which has received such preliminary treatment may be filtered through slow sand filters at rates from 4,000,000 to 8,000,000 gal. per acre per day, or considerably higher than the 2,500,000 to 3,000,000 gal. per acre daily at which they are usually operated.

In some cases these preliminary filters have been upward-flow filters, *i.e.*, those in which the water passed upward through layers of gravel, slag or coke and compressed sponge clippings. This type of filter, known as the Maignen filter was installed in a plant (Fig. 21) at South Bethlehem, Pa., in connection with slow sand filters for the treatment of water from the Lehigh River. The latter stream carries at times of freshets a considerable amount of fine culm from coal mines, which gives it a deep black color. To prepare this water for slow sand filtration a scrubber or preliminary filter was placed at the rear of each slow sand filter tank, and was separated from the latter by a low division wall. The water passed upward through the scrubbers and across the division wall on to the slow sand filters.

The supply pipe to the scrubbers is an 8-in. cast-iron pipe connected to a Y in each scrubber. From the Y extend two lines of

6-in. terra-cotta pipe, which are laid on the floor of the filter and at equal distances from the center line and side walls. The terra-cotta pipe is laid with tight joints, but each 3-ft. length is perforated with nine 1-in. holes in three rows, one row on the bottom and one row on each side of it at an angle of 45° with the bottom center-line. A wash-water drain of reinforced concrete of

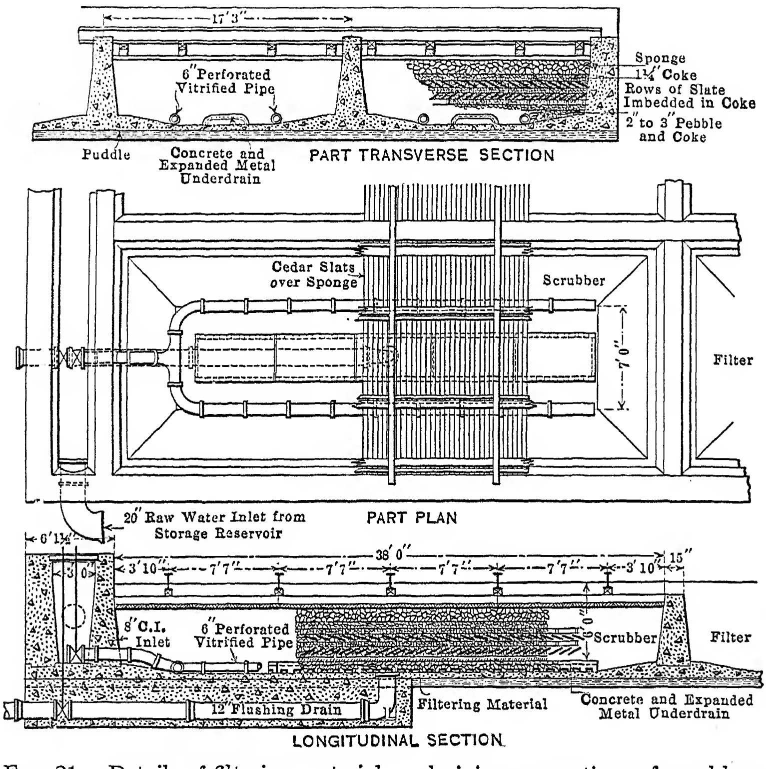


FIG. 21.—Details of filtering materials and piping connections of scrubbers at So. Bethlehem plant.

half elliptical cross-section with its convex side upward is laid along the center line of the filter and connects with a 12-in. drain at the center. Two by 3-in. openings on 7 ½-in. centers along each edge, where the drain rests on the floor of the filter, provide inlets to the drain.

"The space between the supply pipe and the waste water drain is filled with selected 3-in. gravel. The remainder of the lower foot o, filtering material is made up of 3-in. coke. Four layers of 1¼-in. cokef

with an aggregate thickness of 2 ft., are placed over the 3-in. coke. In each layer of the 1¼-in. coke are placed regular rows of slates about the size of ordinary roof slates. The rows in the lower layer are placed longitudinally in the scrubber, and those in the layers above it alternately, transversely and longitudinally. The slates in the lower and upper longitudinal rows are inclined about 30° from zero and 180°, respectively; the same amount and relation of inclination is maintained between the two sets of transverse rows. Over the coke containing the rows of slates is an additional 10 in. of 1¼-in. coke, and over that a layer of sponge, originally 18 in. thick, but compressed to make the total depth of the filtering material in the scrubber 4 ft. 6 in. The mass of filtering material is prevented from floating, due to its buoyancy and the upward flow of water through it, by a grillage of 2 by 3-in. cedar slats placed transversely in the scrubber with ½-in. spaces between them. The cedar slats are held in place by longitudinal 6 by 8-in. yellow-pine stringers wedged against transverse 12-in. I-beams, imbedded at their ends in the concrete of the division and side walls, as shown in one of the illustrations."[1]

The Maignen type of filter was installed in the Lower Roxborough filter plant in Philadelphia, and was constructed essentially as described above for those built at the South Bethlehem plant. At the Belmont plant in Philadelphia there are nine separate filters divided into three compartments each. The first compartment contains ordinary coke, and the water entering at one end at the bottom flows through it horizontally to the bottom of the next compartment. This latter compartment is filled with a layer of sponges about 6 ft. deep. The water flowing upward through this sponge layer next passes to the third compartment, which is filled with coke breeze ranging from ⅛ to ¼ in. in diameter. The water filters downward through this coke breeze at the rate of 40,000,000 gal. per acre per day.

It was found that the first two compartments of these scrubbers were not effective in removing turbidity, and the third compartment is now used only for preparing the water for the final filtration through slow sand filters.

Torresdale Preliminary Filters.—The preliminary filters of the Torresdale plant at Philadelphia (Fig. 22) are a good example of "roughing filters" in which no coagulants are used. They are used to treat the water of the Delaware River preparatory to its filtration through slow sand filters of the usual type. They are

[1] "Water Purification at South Bethlehem, Pa." *Eng. Record*, vol. 52. No. 3.

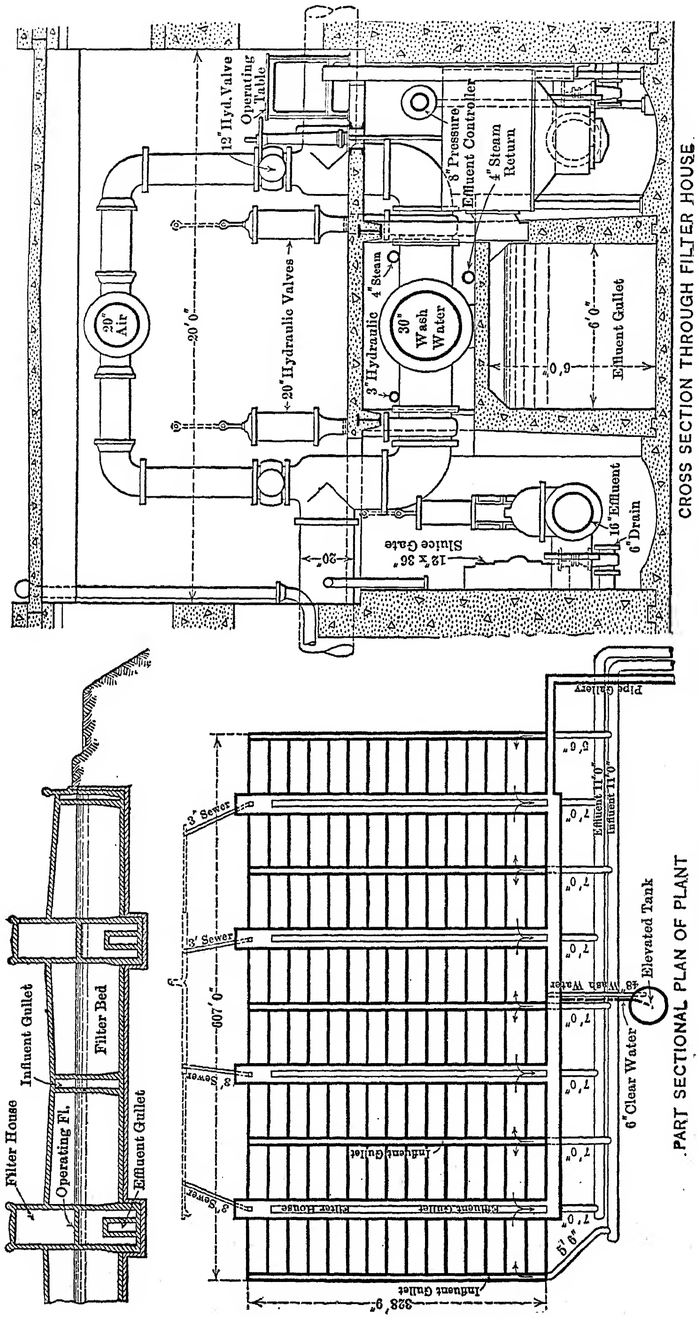


FIG. 22.—Torresdale preliminary filters, Philadelphia, Pa.

in fact mechanical filters of the rapid sand type, differing principally in the character of the filtering layers of sand and gravel.

They consist of a series of 120 beds arranged in eight rows or batteries of 15 beds each. There are four filter houses each running across the entire width of the plant and controlling two batteries of 30 filters each. Each bed measures 20 ft. 3 in. by 60 ft. in the clear, and is controlled from its own individual operating table. The filtering material consists of 15 in. of gravel varying in size from 2 to 3 in. in diameter, 4 in. of gravel varying from 5/8 to 1 1/2 in., 3 in. of gravel varying from 1/4 to 1/2 in., 8 in. of gravel varying from 1/8 to 1/4 in., and 12 in. of coarse sand varying from 0.03 to 0.04 in.

The influent gullets[1] for these filters run the full length of the plant, and serve one or two batteries of beds. They are formed by longitudinal walls at the backs of the beds of adjacent batteries, and are roofed with 6-in. reinforced-concrete slabs. The wash-water gullet is constructed of reinforced concrete 12 in. wide and 4 ft. 3 in. deep. Extending out laterally on each side of this gullet, and terminating at the side walls of a filter are sheet-steel wash-water troughs 18 in. wide, increasing in depth from 6 in. at the wall line to 9 in. at the gullet. Four feet of water is carried over the filter bed when in operation.

Two collectors extend the full length of each bed and on each side of the wash-water gullet; they are constructed of reinforced concrete rectangular in section, 30 in. wide and 8 in. deep, inside measurements. After passing through a hydraulically operated valve and an effluent controller, the water passes to the effluent gullet, which is of reinforced concrete rectangular in shape and is located under the operating gallery of each house. Thirty-inch wash-water mains are laid on top of the effluent gullets, and from these the wash-water piping is led into each filter through 20-in. spiral-riveted galvanized pipe suspended from the roof and connected at the center of the bed and directly above the wash-water gullet with four 12-in. pipes diverging toward the corners of the bed. These diverging pipes in turn each connect with two 8-in. pipes directly above the filtered-water collectors, and from these 8-in. pipes vertical downtakes of the same diameter connect with a manifold. There are eight manifolds in each filter which have 1 1/2-in. galvanized laterals with 3/16-in. holes in the bottom ex-

[1] "Description of Filtration Works and Pumping Stations of Philadelphia." Bureau of Water, Philadelphia, Pa.

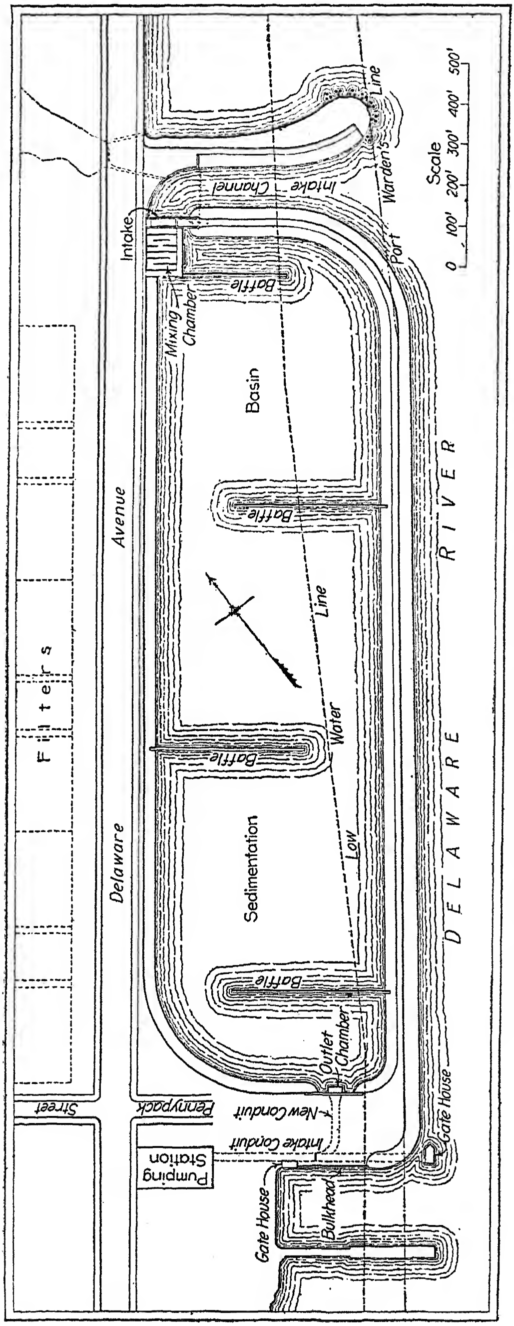


FIG. 23.—New sedimentation basin, Torresdale filter plant, Philadelphia.

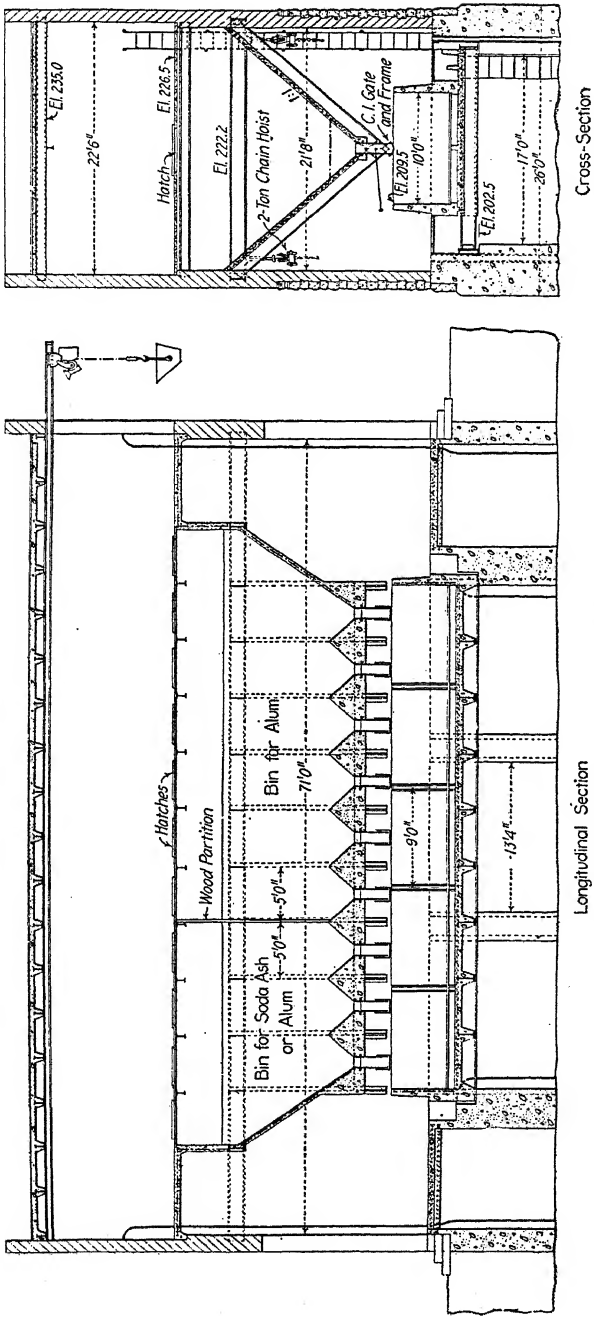


FIG. 24.—Arrangements for treating water with chemicals at Torresdale filter plant, Philadelphia.

tending on 5¾-in. centers. The holes are likewise placed on 5¾-in. centers. These holes are brass-bushed.

The wash water is drained from the wash-water gullet through a 12 by 36-in. hydraulically operated sluice gate discharging into the wash-water drain, which simply consists of an open space between the wall of the filter bed and the side of the effluent gullet. The main air supply consists of a 20-in. pipe running the full length of each filter house and is suspended from the roof. The air system is connected to the wash-water piping and air is introduced through the manifold in the bottom of each filter.

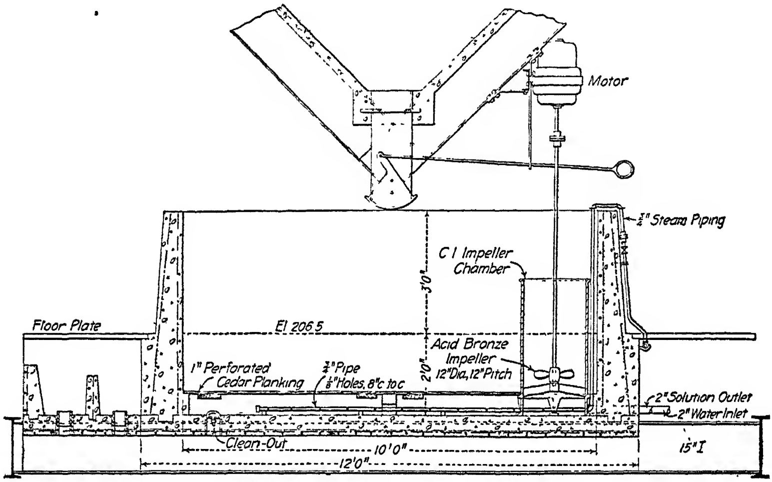


FIG. 25.—Arrangements for treating water with chemicals at Torresdale filter plant, Philadelphia.

The agitation of the water in the filter bed by compressed air precedes the washing with water. Periods of service between washings may vary from 12 hr. to 4 days. The usual rate of filtration is 80,000,000 gal. per acre per day. The rated capacity of the plant is 240,000,000 gal. per day.

The difficulty encountered at the Torresdale plant in obtaining at all times sufficient clarification of the raw water with the roughing filters alone finally led to the construction of a sedimentation basin in which coagulants might be used if desired.

The Puech-Chabal System.—Water-purification plants of this type make extensive use of preliminary filters prior to applying the water to slow sand filters. A number of plants have been installed near Paris and in other European cities.

In this system the raw water first passes through a set of three or more roughing filters of coarse-graded gravel, which varies

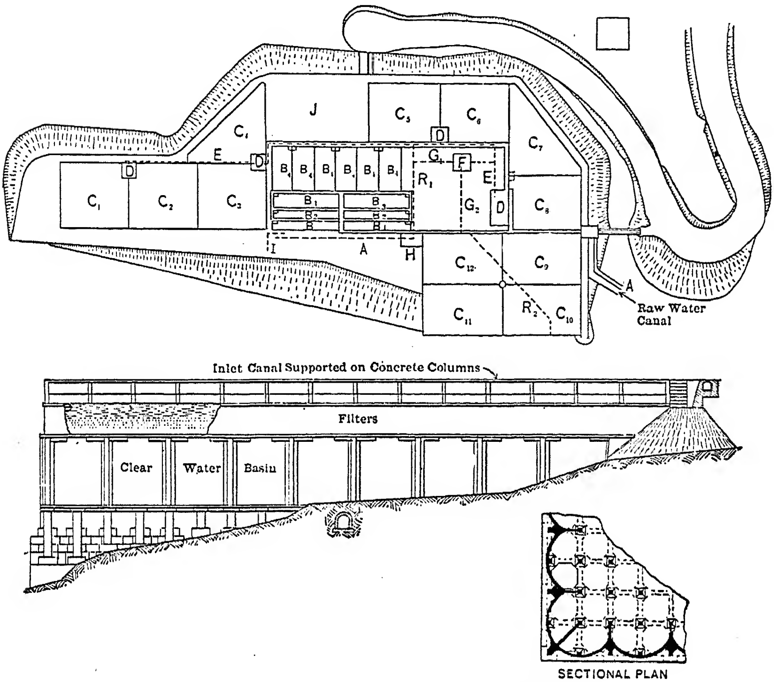


FIG. 26.—Puech-Chabal system of filtration.

in size from 1 in. in the first basin to smaller sizes in each succeeding basin. By graduating the sizes of the material in these

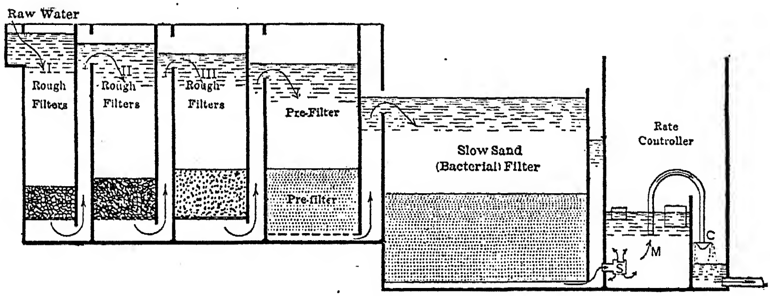


FIG. 27.—Puech-Chabal system of filtration.

roughing filters the velocity of the water through them is greater in the first basins than in the last.

The partially clarified effluent from the roughing filters passes through a so-called pre-filter and from thence to a slow sand filter of the usual type.

This system has been described in some detail by Mr. William F. Johnston in the *Engineering Record*, May, 6, 1911. The accompanying cuts are illustrative of the process.

Operating Results and Costs of Preliminary Treatment.—The results obtained in the operation of plants for the preliminary treatment of water for slow sand filters depend upon the adequacy of the particular method employed. Variations in the

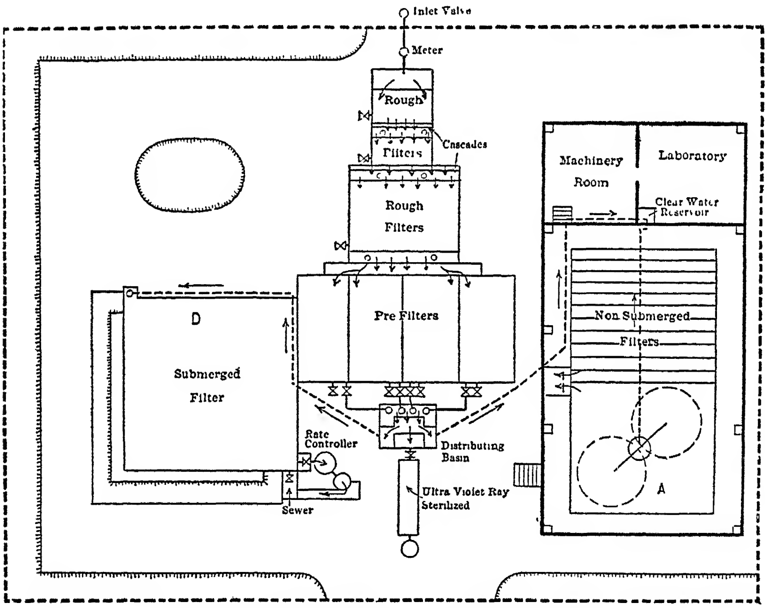


FIG. 28.—Puech-Chabal system of filtration.

amount and kind of sediment to be handled often tax such plants severely, and naturally the slow sand filters have to bear sometimes more and sometimes less than their proportion of the burden.

At the Washington purification plant the benefits obtained by preliminary treatment with aluminum sulphate have been partly indicated in the preceding pages. The following table shows quite clearly the great aid this preliminary treatment is to the slow sand filters in reducing both the bacteria and sediment which they are called upon to remove.

BACTERIA AND TURBIDITY AVERAGES—JANUARY TO JULY

	1906–1911		1911		1912		Days alum was added	
	Turbidity	Bacteria	Turbidity	Bacteria	Turbidity	Bacteria	Turbidity	Bacteria
Great Falls	128	.	70	. .	302	. . . .	678	. .
Dalecarlia inlet.	. . . .	9,000	39	8,735	135	12,707	302	22,534
Dalecarlia outlet[1]	62	7,043	26	6,387	65	9,367	120	15,335
McMillan outlet	30	3,242	14	2,357	11	1,769	10	1,310
Filtered water ..	3	73	0+	46	0+	60	0	28

[1] Water flows through Dalecarlia Reservoir to its outlet, at which point it receives the solution of aluminum sulphate Some alum was used in winter months of 1911, but regular plant for applying coagulant was not in operation until 1912.

The amount of aluminum sulphate used between the first of July, 1911, and June 30, 1912, was 534.49 tons, and was applied on 110 days in this period. The average quantity of aluminum sulphate added to each gallon of water was approximately 1 grain. The cost of this preliminary treatment was $0.53 per million gallons filtered, of which $0.48 was for the chemical and $0.05 for labor.

The Torresdale preliminary filters in Philadelphia were placed in operation on Jan. 21, 1909. The daily average reductions in turbidity and bacteria for the year 1910 were 67.4 and 68.5 per cent., respectively. The average turbidity of the water applied to the pre-filters was 25 parts per million, and the maximum 260 parts per million. The percentage of wash water used averaged 1 per cent. of the amount filtered. Each pre-filter was cleaned on an average of 334 times during the year, or the period of service between cleanings averaged about 1.1 days.

The cost of operating the Torresdale preliminary filters was $0.30 per million gallons. This does not include any charge for laboratory supervision. The operating costs per million gallons for preliminary filtration through the Maignen type of filters at the Lower Roxborough and Belmont plants in Philadelphia were $1.23 and $0.53, respectively.

The Lower Roxborough filters effected an average removal of turbidity of 51.4 per cent., and of bacteria 41 per cent. The Belmont pre-filters reduced the turbidity 50.1 per cent., and the bacteria 44.5 per cent.

At Wilmington, Del., preliminary filters of the Maignen type

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showed the following efficiencies in operation for a series of years ending June 30, 1912:[1]

Year	Unfiltered water	Effluent from preliminary filters	Per cent removed
	Parts per million		
1907–08	24 8	10 6	57.26
1908–09	33 2	11 8	64.46
1909–10	30 6	14 6	52.25
1910–11	49 0	20 0	59.18
1911–12	68 0	36 0	47.06
Average.	41 3	18 6	54 96

The average reduction in color by the pre-filters was 2.3 parts per million of the standard scale in 1912, or from 24.5 parts to 22.2 parts per million. This is a removal of 9.38 per cent. of the total color. The range in color in the raw water was from 70 parts per million to 17 parts of the standard scale.

The efficiency of the pre-filters in removing bacteria is shown in the following table:

Year	Unfiltered water	Effluent from preliminary filters	Per cent. removed
	Bacteria per cubic centimeter		
1907–08	5,116	3,545	30.71[2]
1908–09	4,142	2,171	47.79[2]
1909–10	5,248	1,250	76.19
1910–11	31,955	16,256	49.13
1911–12	49,533	29,816	39.06
Average.........	15,999	8,840	44.75

N.B. The average efficiency of these filters judged from counts made from cultures on agar as a medium was 39.29 per cent., while with gelatin the percentage removal indicated was 54.79 per cent.

The preliminary filters showed a reduction of 4.94 per cent. in the number of positive tests for the Bacillus coli during the year 1911–12.

[1] Report Board Water Commissioners of Wilmington, Del., 1911–12.

[2] Agar counts.

The cost of operating the Wilmington preliminary filters is given in detail in the following table:

	Total cost	Cost per million gallons
Salaries	$863.25	$0.21
Additional labor	10 63	0.005
Supplies	2 34	0.001
Repairs and renewals	96.41	0.023
Light and power	511.55	0.124
Other operations	51.21	0.012
Special	688 30	0 166
Total	$2,223.69	$0.541

The above figures are gross operating costs but do not include interest or depreciation on plant investment. The item "Special" refers to improvements on the filter beds, and if omitted would bring the cost down to $0.375 per million gallons.

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1. George W. Fuller: "Report on Water Purification at Cincinnati, Ohio."
2. F. F. Longley: "Experiments on Preliminary Treatment of Potomac River Water at Washington, D. C." *Eng. Record*, vol. 57, June 27, 1908.
3. Wm. F. Wells: "Some Notes on the Use of Alum in Connection with Slow Sand Filtration at Washington, D. C." *Proc.* Am. Waterworks Assn., June, 1913.
4. G. A. Johnson: "Preliminary Treatment of Water for Slow Sand Filters at Pittsburgh, Pa." *Eng. News*, vol. 68, Oct. 3, 1912.
5. "Water Purification at South Bethlehem, Pa." *Eng. Record*, vol. 52, No. 3.
6. "Description of Filtration Works and Pumping Stations of Philadelphia." Bureau of Water, Philadelphia, Pa.
7. Report Board of Water Commissioners of Wilmington, Del., 1911–12.
8. W. F. Johnston: "Water Purification by the Puech-Chabal System." *Eng. Record*, vol. 63, May 6, 1911.

CHAPTER XI

SYSTEM OF SLOW SAND FILTRATION

The most efficient method for the final treatment of a water, which contains finely divided material in suspension and one which has proven entirely satisfactorily in practice, is to subject it to filtration through a bed of sand. Other filtering mediums may be used; but the small size of the particles of which the sand is usually composed, which makes it possible to prepare a compact yet porous bed, its slight deterioration by usage and the relative ease with which it may be cleaned, its wide distribution, and its low cost, are the reasons for its general employment. Sand filtration, as commonly practised, may be divided into two general methods, depending on the rate at which the water flows through the sand bed. These rates of filtration are so widely different that a classification as slow sand filters and rapid sand filters is sufficient to characterize the two systems. Where the rate of filtration does not exceed 6,000,000 to 8,000,000 gal. per acre per day, the filters are called slow sand filters. Rates as low as 1,500,000 gal. per acre per day are used, but the average rate is probably between 2,500,000 and 3,000,000 gal. per acre per day. Under some very favorable circumstances the yield may be as high as 6,000,000 or 8,000,000 gal. per acre per day. In such cases the unpurified water is usually quite free from suspended matter, or preliminary treatment of some sort has been employed.

The rates at which rapid sand filters are operated vary widely, the usual rate being about 125,000,000 gal. per acre daily. The rate may be 40 per cent. lower or 20 per cent. higher than the above-named rate and still be regarded as a rapid sand filter. Rapid sand filters almost invariably require the water to be prepared for such high rates of filtration by being treated with some agent which will coagulate the impurities suspended in the water, and which will furnish material in a colloidal condition that will cover the surface of the sand as soon as the water begins to pass through the bed. In this class of filters the large volume

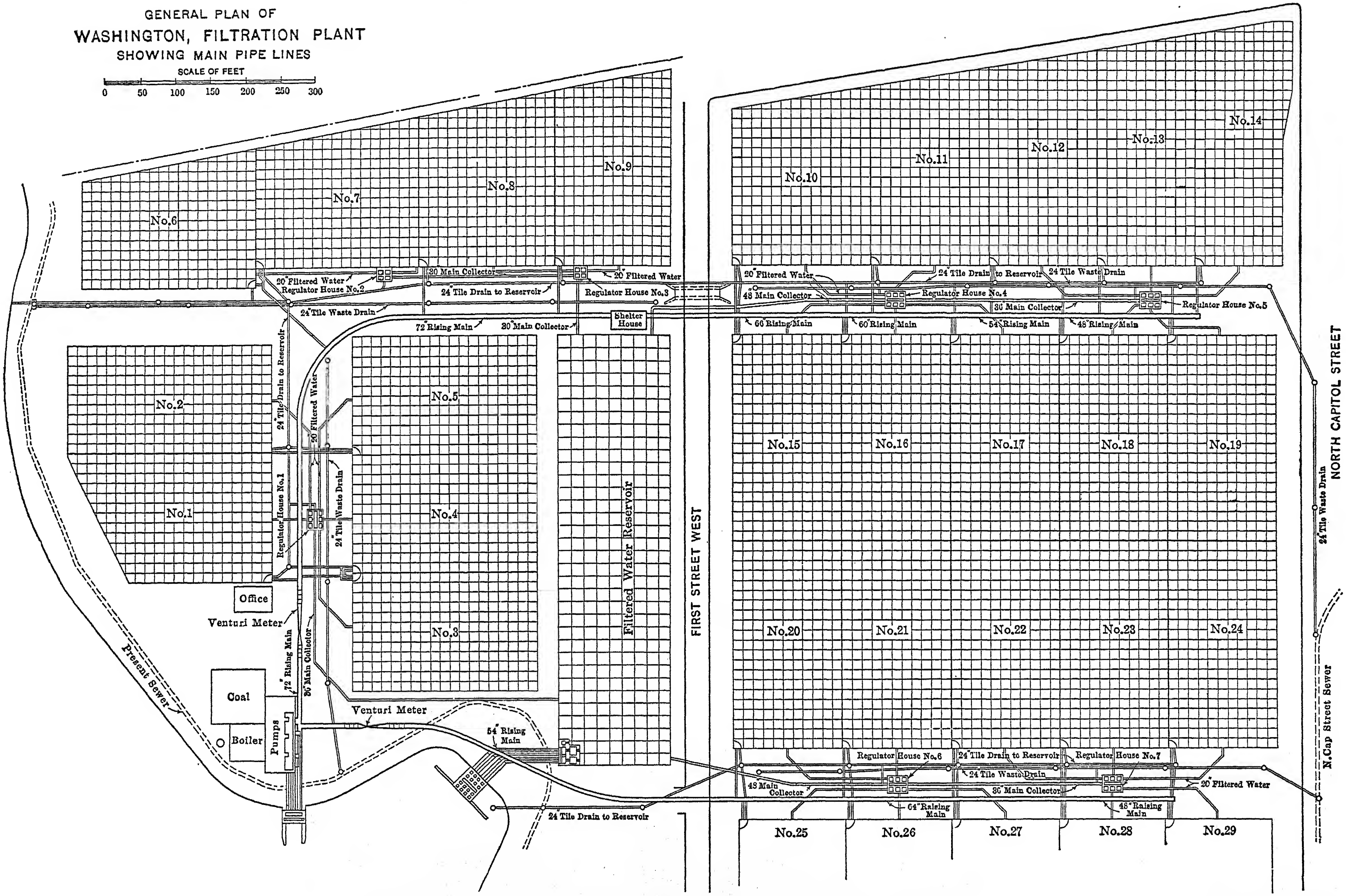

Fig. 29.—General plan of Washington filtration plant showing main pipe lines.

(*Facing page* 116.)

of water, which passes through the bed in a comparatively short period of time, makes it possible to build them of relatively small area as compared with slow sand filters, and also necessitates efficient and rapid methods of cleaning. These filters will be considered in detail in subsequent chapters.

General Form and Construction of Slow Sand Filters (Figs. 29, 30, 31 and 32).—If the rate of filtration is as low as 2,500,000 or 3,000,000 gal. per acre daily, it is obvious that in order to obtain a sufficient quantity of water for a public supply, a considerable area of sand must be used for filtering the water. In consequence the slow sand filter plants consist of beds of sand varying in size from ¼ to 1 acre in extent. As many beds are provided as may be necessary to supply the volume of water needed at times of maximum draft, with proper allowances made for those beds which may be withdrawn from service for the purpose of cleaning or repairing them.

The sand beds are built both with and without covers. Uncovered beds during the winter months may be operated with difficulty, if the temperature is low enough to form ice in the water on the top of the filter. According to Mr. Allen Hazen, where the normal mean January temperatures are lower than 32°F., it is probably safer to provide covers.

The form and general method of construction of the filter beds depend upon the particular conditions which may exist at the site selected. In form they are usually rectangular, but may take other shapes best adapted to the land available. As these filters are virtually shallow reservoirs and cover considerable area, it is necessary that the embankments and bottom below the underdrains be made water-tight. Paved earth embankments, masonry or reinforced-concrete walls are used for the sides of the filter. The bottom is constructed of either a thin layer of concrete, or of a pavement over a puddle layer, or of inverted groined arches of concrete carrying piers to support a cover for the filter. If the bottom of the filter is not water-tight, troubles may arise from the infiltration of ground water, when the level of the latter is higher than the water on the filters. Loss of water may also result by leakage either to adjoining filters or to some point outside the filters should the water level in the latter be higher than the ground water. According to Mr. Allen Hazen, leaky filter bottoms have probably caused more annoyance in operation than perhaps any other single defect.

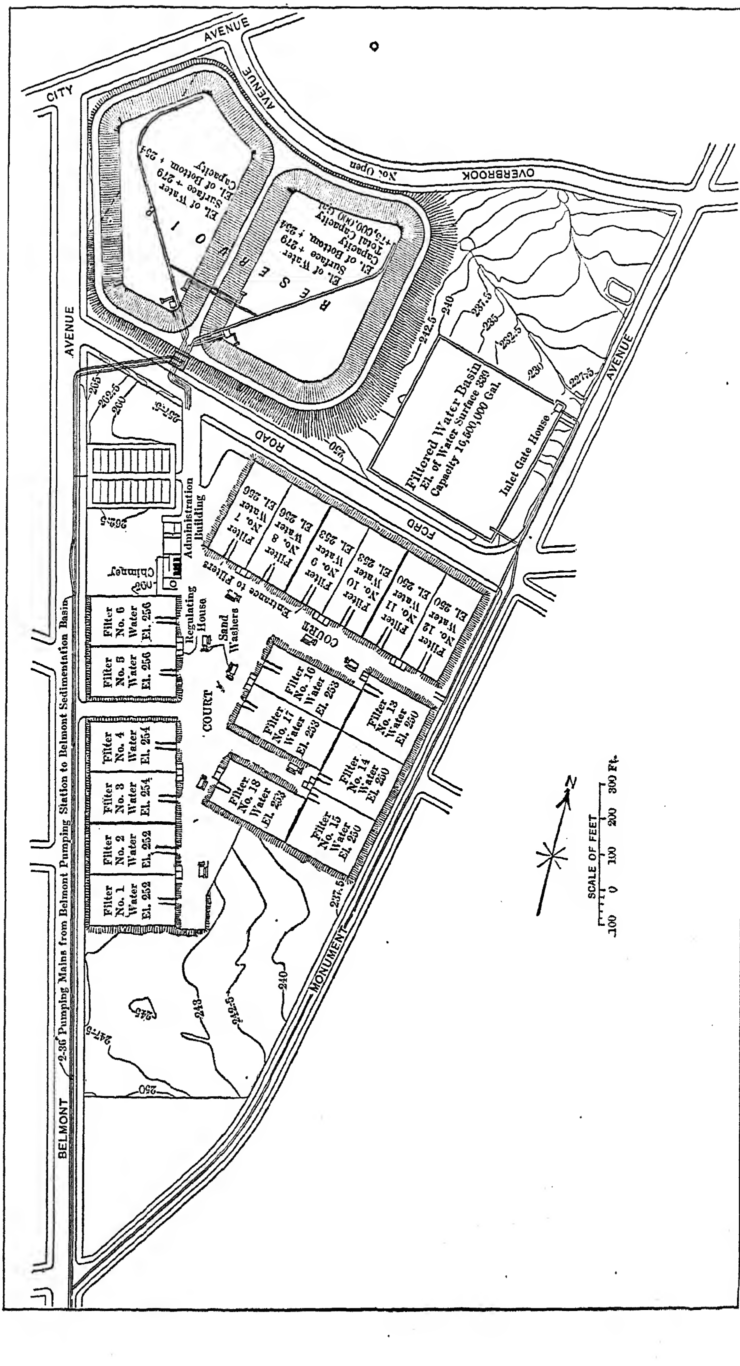

FIG. 30.—Belmont slow sand filters, Philadelphia.

Cracks in division walls and side walls may also cause loss of water and become troublesome in operation.

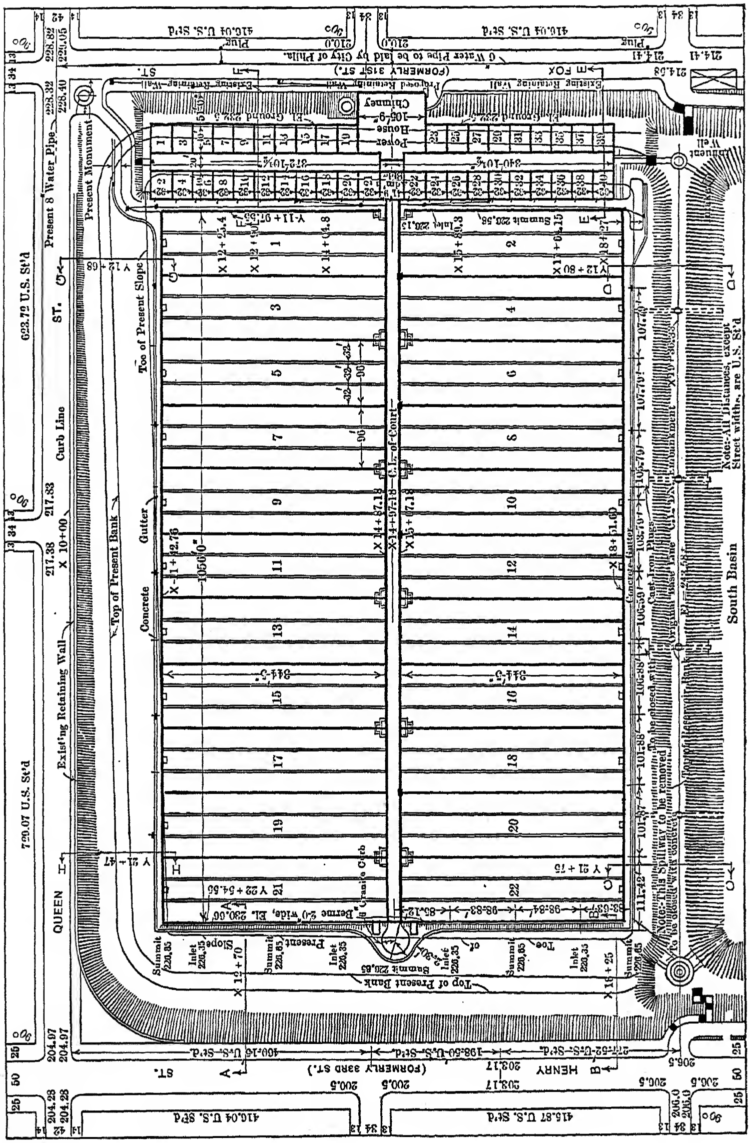

Fig. 31.—Plan of Queen Lane slow sand filters, Philadelphia.

The covers for slow sand filters are constructed of brick or concrete supported by pillars. The groined-arch construction is the

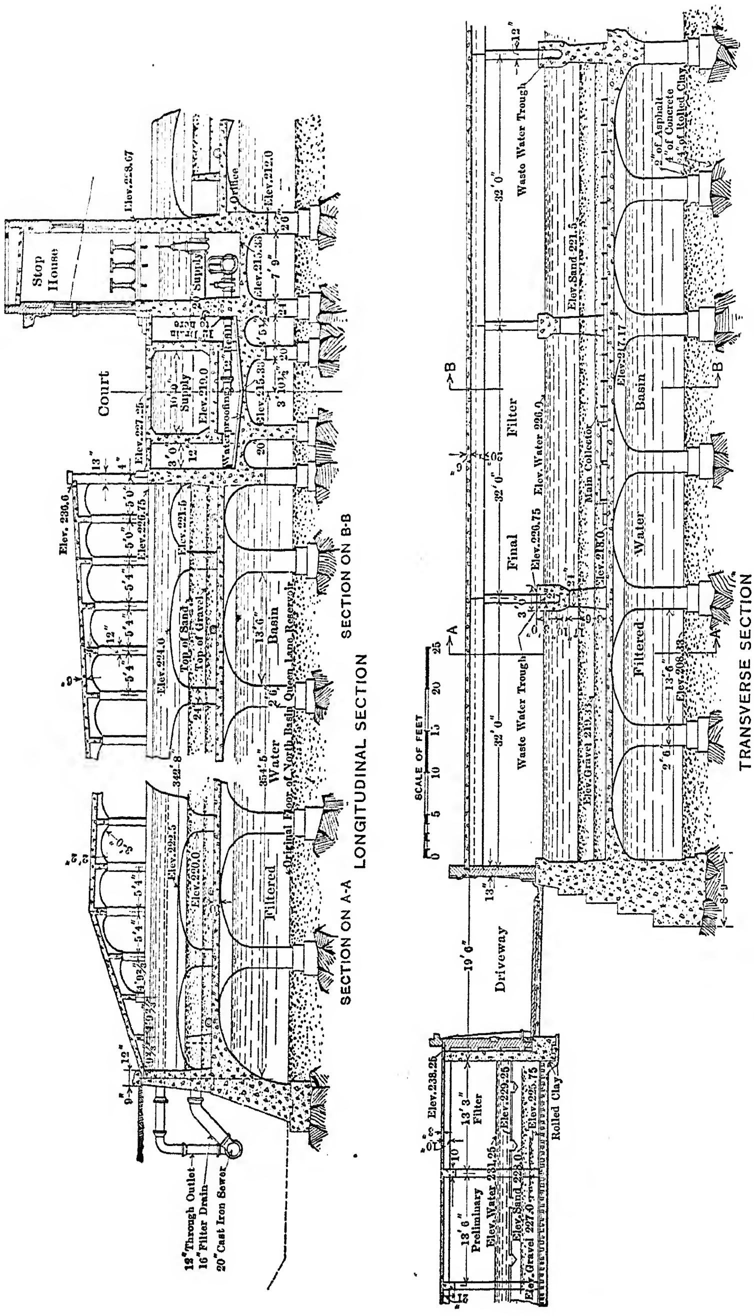

FIG. 32.—Sections through Queen Lane slow sand filters, Philadelphia.

more common form of building the cover, although flat reinforced-concrete covers are used as well. Over the vaulting is spread a layer of earth from 2 to 3 ft. in thickness. The top is provided with openings for the admission of light and air when needed during cleaning, and with entrance galleries for the removal or introduction of sand.

Underdrain System.—The underdrain system which supports the gravel and sand of the filter bed is usually constructed of brick and tile, and is so arranged as to collect the water from all parts of the bed as uniformly as possible. The main drain passing through the center of the bed is fed by laterals which usually enter at right angles to the former. The main drain is often constructed of brick or concrete, but the laterals are usually of tile. The drains are built either above, partly above, or flush with the floor of the filter. If placed on the floor of the filter, their tops do not extend above the coarsest gravel in order to prevent the open joints becoming clogged with small gravel or sand. Some double bottoms of brick have been used in European filters, in which small arches or other convenient method of supporting a false bottom of open brickwork are employed. On this false bottom rest the gravel and sand.

The frictional resistance to the flow of water in the underdrains should be very low, and so distributed as to avoid as much as possible unequal rates of filtration in different portions of the bed. Mr. Allen Hazen suggests that velocities of flow in lateral tile drains between 4 and 12 in. in diameter should be from 0.3 to 0.5 ft. per second, and that in the main drains 0.55 ft. per second should not be exceeded, where the rate of filtration is about 2,570,000 gal. per acre per day. In order to reduce the size of the center drain, and to maintain more uniform rates of filtration throughout the bed, compensating orifices were used by Messrs. Hazen and Hardy in constructing the Washington, D. C., slow sand filters. These orifices consisted of circular openings cut in brass disks, and which were placed at the entrance of the laterals to the main drain. The object sought was to introduce frictional resistance at these points "equal to the combined frictional resistance and velocity head in the main drain from the most remote point to the point in question."[1]

Gravel Layers.—The gravel to be used in slow sand filters must be carefully selected and placed. Only gravel composed of

[1] *Trans.* Amer. Soc. C. E., vol. 57, p. 307.

the harder rocks should be used, *i.e.*, limestone or other easily crushed rock should be avoided. The manner in which the underdrains are laid, and the nature of the floor of the filter determine in a large measure the thickness and way in which the layers of gravel of various sizes are distributed. Around the drains the largest stones are placed, and above them successive layers of intermediate sizes of gravel until the sand is reached. At the Albany, N. Y., filter plant, three grades of gravel were used, the largest size being material which passed a screen having holes 3 in. in diameter, and which was held by a screen having holes 1 in. in diameter. The next size of gravel was that which passed the 1-in. holes and was held back by a screen with 3/8-in. holes; and the smallest size of gravel that which passed the 3/8-in. holes and was retained by a screen with 3/16-in. holes. The largest size of gravel was placed entirely around the 6-in. pipe drains, and slightly above their tops. In some cases it even covered the entire floor of the filter, but this was not uniformly carried out. The next smaller size of gravel was so laid as to bring the gravel layer to the same level all over the filter, and averaged about 2 in. in thickness immediately above the drains. At other points it was somewhat thicker, depending upon the amount of coarse gravel underneath. On top of this second layer was placed a third layer of the finest gravel to an average depth of 2½ in.

In many of the European plants much deeper gravel layers have been used than that described above. Deeper layers are of no particular advantage, since there is little likelihood of the finer gravel and sand getting down into the drains, if the layers are properly graded. In general, if any supporting layer of gravel does not exceed by more than three or four times the average size of the stones of the next layer above it, there is little danger of the finer material penetrating to the lower layers. Each layer must be complete in itself. The gravel should be washed free from sand, earth or other fine material, as a very little matter of this character may clog the gravel layer and produce unequal rates of flow through the bed.

The total depth of gravel layers varies considerably in practice, but is usually from 2 to 3 ft. in most of the European filters. Recent American practice has cut this depth down to 12 and 16 in. Even 6-in. gravel layers, when carefully graded and placed, have proven entirely satisfactory. A gravel bed composed of

6 in. of stones from 1 to 3 in. in diameter, 2 in. of smaller stones from $\frac{1}{2}$ to 1 in. in diameter, 2 in. of coarse gravel with stones from $\frac{1}{4}$ to $\frac{1}{2}$ in. in diameter, and 2 in. of a very coarse sand with particles from $\frac{1}{32}$ to $\frac{1}{16}$ in. in diameter, should, if carefully laid, provide an excellent foundation for the sand bed above it.

In the Washington, D. C., filters, the gravel was placed on the inverted groined arch floor in a manner similar to that at Albany, N. Y., and approximated 12 in. in depth midway between the piers, and decreasing to 3 in. at the piers. Crushed trap rock or granite was used in three sizes, the lower layer of coarse gravel being about 7 in. thick, and the two finer layers each $2\frac{1}{2}$ in. thick. The gravel was kept 2 ft. away from the side walls.

Somewhat deeper layers of gravel were used in several of the Philadelphia filter plants. In these plants gravel ranging in size from $1\frac{3}{4}$ to 3 in. in diameter was placed around the collectors and for a height of 6 in. from the floor of the filter. Above this layer was placed 4 in. of gravel ranging from $\frac{5}{8}$ to $1\frac{3}{4}$ in. in diameter. On top of this was a 3-in. layer of gravel with stones ranging from $\frac{1}{4}$ to $\frac{5}{8}$ in. in diameter, and above the latter a layer of 2 in. of fine gravel of a size from $\frac{1}{4}$ in. in diameter to material retained on a sieve with 14 meshes to the linear inch. On top of the 2-in. layer was a 1-in. layer of coarse sand, which passed a 14-mesh sieve and was retained on a 20-mesh sieve. The total depth of the gravel and coarse sand was 16 in.

Sand.—The large amount of sand required for slow sand filters necessitates the use of the sands usually found in river beds, or on the shores of the ocean or great lakes, or in deposits at or near the earth's surface. Not all of this material is suitable in its composition, and frequently not so as regards the size of the particles. It usually is composed of quartz and hard silicates whose grains are sometimes quite irregular in shape, and with sharp angles. On the seashore, and wherever erosive action of the water has rounded the particles, a much less angular grain is obtained, and one which may be considerably finer than the bank sands. Natural sands are usually mixed with more or less finer material like clay, which should be removed by washing before being used.

Sands for filter purposes should be practically all quartz, or silicates which do not break down easily, and should not contain carbonates of lime or magnesia, except in very small amounts. The presence of aluminous and calcareous material tends to increase the frictional resistance of the sand to the flow of the

water, and salts of lime and magnesia to harden it. Hence specifications for providing filter sand usually set limits for the amounts of these materials, which if exceeded, renders the sand unacceptable.

Size of Sand.—The sizes of the sand grains are an important factor in the operation of a filter, since they materially influence the rate of filtration. In natural sands the relative amounts of the various-sized particles differ considerably. As far as the transmission of the water is concerned, the influence of the finest sand grains is far greater in proportion to their number than are the coarser particles. By filling the voids between the larger grains the finer particles may render an otherwise coarse sand so compact as to offer a high resistance to the flow of the water. It is evident, therefore, that it is the size and number of the pores between the grains of sand, or in other words, the porosity of the sand bed which governs primarily the rate of flow.

As the rate of flow is a function of the size of the sand grains and of their compactness, it is customary to express the velocity of flow in terms involving the so-called "effective size" of the sand grains, together with other factors, such as the head of water, thickness of the sand layer, and temperature of the water. Mr. Allen Hazen's[1] experiments on filter sands led to the development of the following formula:

$$V = cd^2 \frac{h}{l} \left(\frac{t + 10^\circ}{60^\circ}\right)$$

where V = velocity of water in meters daily in a solid column;
c = a coefficient varying from 700 to 1,000 for new and clean sand of fairly uniform size, to as low as 400 for old compacted sand;
d = effective size of sand;
h = head of water;
l = depth of sand layer; and
t = temperature of water in degrees Fahrenhelt.

The effect of the temperature on the rate of flow is quite marked, the rate decreasing or increasing with the fall or rise of the temperature. For example, twice as much water will pass through a given sand at 74°F. as will at 32°F.

Effective Size of Sand.—For a uniformly sized sand of a certain compactness, it has been found that the velocity of flow is nearly

[1] ALLEN HAZEN: "The Filtration of Public Water Supplies."

proportional to the square of the diameter of the sand grain, but as natural sands are not uniform, it is necessary to introduce a coefficient into the formula. Moreover, since the finer particles of a natural sand appear to be the sizes of grains which most materially influence the rate of flow, it becomes a question as to what size shall be chosen to represent the sand as a whole in calculating its frictional resistance. Hazen decided from his experiments that the size of grain such that 10 per cent. of the particles are smaller and 90 per cent. are larger than this size, should be regarded as the "effective size." The ratio of the size of the grains, such that 60 per cent. of the sand is finer than this size to the "effective size," is termed the "uniformity coefficient," or in other words a ratio of the "large particles" to the "small particles" in a sand as above defined.

A very fine sand for use in a filter would be one that had an effective size as low as 0.17 mm., and a very coarse sand one with an effective size as high as 0.50 mm. The more usual sizes for slow sand filters range between 0.20 mm. and 0.40 mm. in effective size. Natural sands will range from 1.5 to 2.5 in their coefficients of uniformity, but the best sands for filter purposes should be less than 2.0.

Mechanical Analysis of Sand.—Sand may be separated into its different-sized particles by means of a series of sieves having meshes varying in number from 10 to 200 per linear inch. Coarser sieves are used for the very coarse sands and the finer gravels. For sands finer than 0.10 mm. (sieve having about 200 meshes per linear inch would separate this size), elutriation methods must be used, as it is difficult to make a much finer sieve than one with 200 meshes per inch with which results of any practical value can be obtained.

By sifting a known weight of a dried sample of sand through a series of graded sieves, it is possible to calculate from the weights of sand left on the different sieves the proportion retained to that passed by any given size of mesh. Because of the variation in the sizes of the openings in the wire cloth of any given mesh, Mr. Allen Hazen derives the size of separation of a sieve by taking the last particles of sand passing through it, and determining their weight and specific gravity. Assuming each particle to have a diameter of a sphere of equal volume, and taking the specific gravity of a quartz sand as 2.65, it is possible to calculate the mean diameters of the particles from the above

data. Where d represents the diameter of the particle in millimeters, and w its weight, then the size of separation may be obtained from the formula:

$$d = 0.9\sqrt[3]{w}.$$

From actual measurements of the openings in the mesh by means of a microscope, using stage and ocular micrometers to obtain accurate readings, Mr. Philip Burgess obtains the size of separation of a sieve by multiplying the width of the space by a factor derived from a comparison with results obtained by Hazen's method. This factor is 1.095 for woven-wire cloth, and 1.19 for twilled-wire cloth. These factors were obtained after counting over 250,000 sand grains, which had been separated by Hazen's method of sifting, and then determining their weight and specific gravity.

By a sufficient number of separations the percentages by weight of the sand smaller than the various sizes may be plotted to a uniform or to a logarithmic scale. From this curve intermediate points may be determined with a fair degree of accuracy, and the relative proportions of the variously sized particles be seen at a glance. From such a curve the effective size of the sand and the uniformity coefficient are most conveniently determined.

Specifications for filter sand may be and usually are quite explicit, as the following specifications for supplying sand to the slow sand filters at Providence, R. I., will show:

"The filter sand shall be clean bank sand, or equal, screened and washed, as may be necessary to secure the requisite cleanliness and grain sizes, and shall be entirely free from clay, dust, roots and other impurities. The grains shall, all of them, be of hard material which will not disintegrate, and shall be of the following diameters: Not more than one (1) per cent. less than thirteen-hundredths (0.13) of a millimeter, nor more than ten (10) per cent. less than twenty-six hundredths (0.26) of a millimeter. At least ten (10) per cent. by weight shall be less than thirty-four hundredths (0.34) of a millimeter; at least sixty (60) per cent. less than eighty-three hundredths (0.83) of a millimeter, and at least eighty-five (85) per cent. less than two and ten hundredths (2.10) millimeters. No particle shall be more than five (5) millimeters in diameter, and the sand shall be passed through screens and sieves of such mesh as to stop all such particles, and no screen shall be used containing at any point holes or passages allowing grains

five (5) millimeters to pass. The diameter of sand grains shall be computed as the diameters of spheres of equal volumes."[1]

When sand is required to be washed, because of the clay or other fine soil it may contain, specifications have sometimes required that the washing shall be continued until a definite weight of the washed sand when shaken with a certain volume of clear water, does not produce more than a certain turbidity, measured according to the accepted silica standard scale. At Pittsburgh, Pa., it was required that 100 grams of the washed sand when shaken with 1 liter of water should not cause a turbidity greater than 200 parts per million. At Washington, D. C., a turbidity was not permitted which exceeded about 0.2 per cent. of clay in the washed sand, or about 4,000 parts per million of turbidity.

At Providence the sand specifications stated: "the filter sand shall not contain more than 1 per cent. of lime and magnesia soluble in dilute hydrochloric acid in 24 hr. at 70°F., taken together and calculated as carbonates." At Albany, N. Y., 2 per cent. was made the limit for lime and magnesia and calculated as at Providence.

Depth of Sand.—Since the sand bed offers the greatest resistance to the flow of the water, its depth should not be greater than will provide good purification of the water being filtered. Beds of fine sand may be of less depth than those of coarse sand, but the former produce greater losses of head than do the latter, and are likely to clog more quickly. With the average sand of 0.35 mm. effective size, and a uniformity coefficient of 2.0, the depth may be from 30 to 40 in. This depth will allow for removals of more or less sand by scraping for the purpose of cleaning the filter bed. Deep beds of sand are likely to be less adversely affected by changing rates of flow or by scraping than shallow beds, and are, therefore, safer and easier to operate. Fine sands permit much less water to pass through them between cleanings of the bed by scraping than do coarse sands. The penetration of the impurities into the sand is also much less in fine sands than in coarse, and for that reason are somewhat more quickly and economically cleaned, since it is necessary to handle less sand when scraping the beds.

In Germany the law does not permit a sand bed of a filter to

[1] Otis F. Clapp: "Description of Slow Sand Filtration Plant, Providence, R. I."

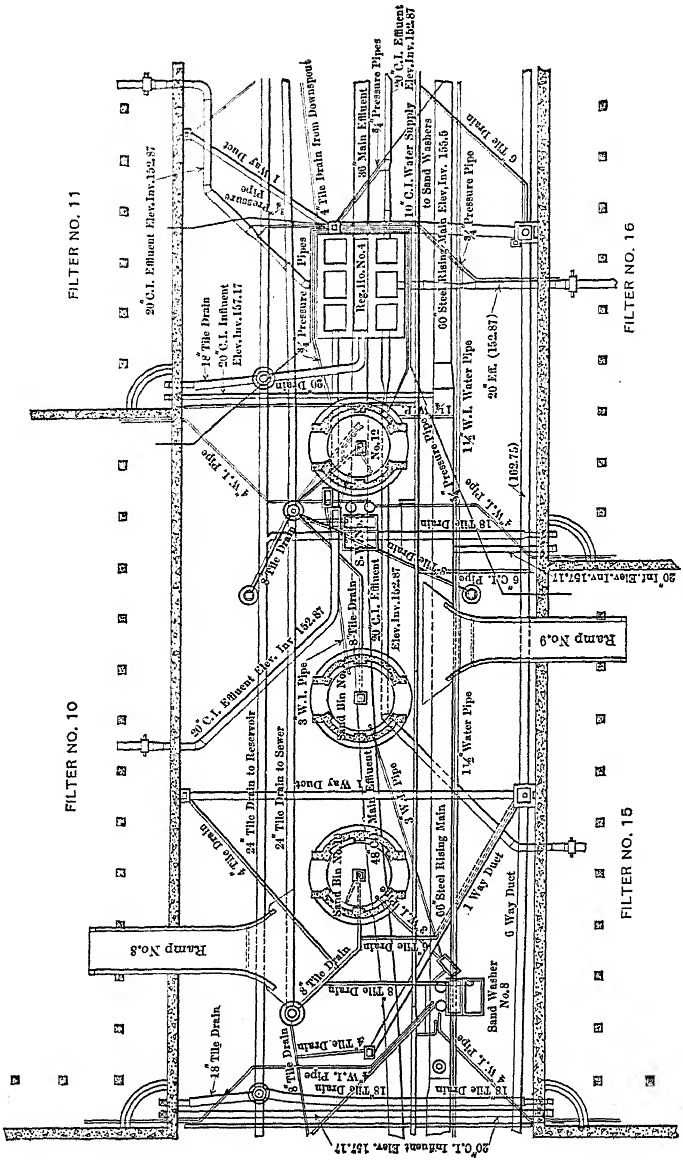

Fig. 33.—Detailed plan of piping in sand court, Washington, D. C., filtration plant.

be used after the sand has been reduced by scraping to less than 12 in. In actual practice, however, both in this country, and in Europe, the sand beds are usually considerably more than 12 in. in depth after the final scraping and before being refilled with sand to their original depth.

Arrangement of Sand Courts and Controlling Chambers.—Slow sand filter (Fig. 33) beds are usually built side by side in two rows facing a common open area between them. In large plants a duplication of this arrangement is carried out where possible, as it affords a convenient place for storing and handling the large amounts of sand which are employed in this process. Economy of construction is also obtained by this arrangement, since a common division wall between adjoining filters reduces the cost of masonry and embankments. Water to be filtered is readily distributed to the sand beds on either side of the court, and effluent piping for the filtered water is also most conveniently located in this same area.

Controlling chambers, in which the piping from two or more filters are brought together, are usually placed on either side or in the center of the court as may be most convenient. Drain piping, sand bins, sand-washing devices and conveying apparatus are necessarily located within this space between the rows of filters.

The use of this space for handling and storing sand has not always proven satisfactory, owing to the insufficient area provided. At the filter plant in Albany, N. Y., the use of the court between the filters has been abandoned for washing or storing sand for this reason. Methods of washing the sand in place, or of immediate transfer of the washed sand to another filter, have obviated to some extent the necessity for sand courts.

A somewhat unusual arrangement of filters exists in the Lower Roxborough filter plant at Philadelphia, where filters were placed in a series of steps owing to the topography of the site. Adjacent filters differ in level by 2 ft. 9 in.

Depth of Water on Sand.—The depth of water over the sand layer varies considerably, but in modern plants ranges from 3 to 4 ft., when the sand is at its maximum depth before scraping of the bed has commenced. The older plants sometimes carry a greater depth of water than 4 ft., but in such cases the method of controlling the outflow from the filter is crude and unreliable. From 5 to 7 ft., of water are carried over the sand at the Pitts-

burgh slow sand filter plant, which seems to be considerably more water on top of the sand than is the usual custom (Fig. 34).

Comparatively simple devices for regulating the inflow of water to the bed are now generally used and, when employed

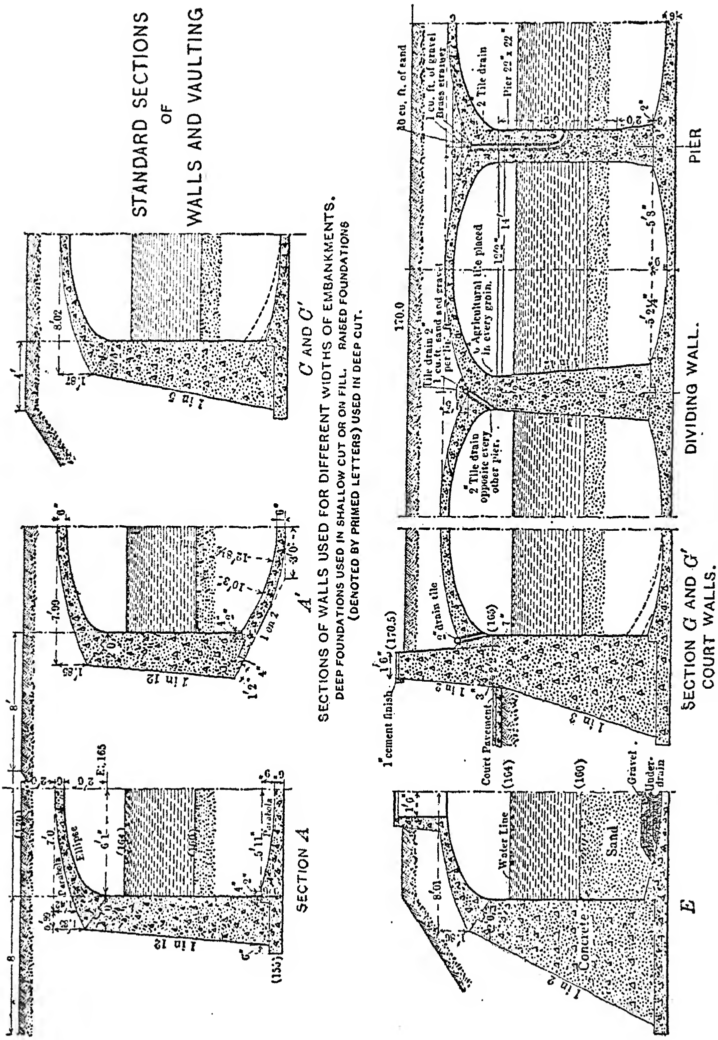

FIG. 34.—Cross-section of filters, Washington filtration plant.

in addition to regulating devices for controlling the rate of discharge from the filter, make it possible to maintain a practically constant depth of water over the sand. With reliable controlling

apparatus there is no reason why in most cases one-half to one-third the prevailing depths of water on the sand may not be used with results equally as good as are now obtained.

Loss of Head.—The resistance offered to the passage of the water through the filter bed varies, being slight at first when the sand is clean, and gradually increasing as it becomes clogged with material strained out of the water and deposited on the surface of the bed and in the passages between the sand grains. The difference in level between the water on the top of the filter bed and the height to which the water will rise after issuing from the underdrains of the filter is the measure of this frictional resistance or the loss of head. The loss of head is measured by means of floats located in chambers connected with the water on the top of the filter, and the water discharging from the underdrains. These floats in their separate chambers are connected to indicating apparatus, which may be placed in any convenient position for easy observation.

Limits to the Loss of Head in Operation.—As the loss of head indicates the extent of clogging in the filter, it furnishes a method of determining how long the filter may be safely operated, provided experience has shown that with any given water an increase in the loss of head beyond certain limits accompanies a deterioration in the quality of the filtered water. The relation between loss of head and quality of water is by no means constant for any given water, and depends upon several factors, such as the character of the material being strained out of the water, the rate and fluctuations in the rate of filtration, the fineness and depth of the sand bed, and the disturbance of the filtering medium at and near the surface of the sand by the escape of air. Carrying the loss of head to a point where the mouth of the effluent pipe is uncovered, will permit air to enter the underdrains and work its way up through the bed. This is not of course permitted in well-operated plants. Air dissolved in the water may also be liberated in any zone of diminished pressure within the sand bed, and in moving upward through the sand bed produces similar disturbances to those caused by air entering the underdrains.

In practice the loss of head permitted, before filters are withdrawn from service for cleaning, varies widely. In some cases only 18 to 24 in. are allowed, while in others 6 or 7 ft. may be reached. The average loss of head is probably from 4 to 5 ft.,

at least in American practice with modern regulating apparatus. In Europe, however, Mr. Allen Hazen noted a strong tendency to limit the loss of head to about one-half the average figures stated above. Only actual tests will in most cases show the loss of head which may be permitted, consistent with maintaining a high grade of filtered water and economical operation of the filters. Too frequent cleanings of the sand beds mean increased cost of maintenance, which can easily be brought about by arbitrarily fixing a limit to the loss of head, above which the filter shall not be operated, without knowing whether a greater loss of head might not be utilized and still obtain a good quality of effluent.

Causes for Deterioration in Quality of Water with High Loss of Head.—In some of the older slow sand filter plants, the filters were connected with the filtered water reservoir in such a way that fluctuations in the water level of the latter, due to increased or decreased consumption of water, produced corresponding changes in the rates of filtration. With clean filter beds fluctuations in the rate of filtering were not especially harmful, but as the loss of head increased the sudden changes in the rate were liable to cause the filters to discharge a poorer quality of water. The theory that the surface films on the sand layer were broken through by the increased pressure due to a high loss of head has been commonly held, especially in Europe. Mr. George W. Fuller[1] in his report upon the operation of the experimental slow sand filters at Cincinnati, Ohio, makes a statement which seems to fit into modern theories of the action of filters as set forth in a preceding chapter. He states that "it may be noted that the increased losses of head occur at times when the *retentive capacity of the sand layer* is more or less exhausted; and, upon the application of very turbid water, the applied clay passes through the portion of the filter where the frictional resistance is the *greatest*."

If the colloidal matter which is supported by the sand grains loses its "retentive capacity" by becoming surcharged with suspended particles, and is no longer able to hold these particles, whether it be by electrical attraction or by purely mechanical means, it necessarily follows that further additions of suspended particles will result in their passing through the minute capillary passages of the bed and their appearance in the effluent of the

[1] Report on Water Purification at Cincinnati, Ohio, p. 189.

filter. This explanation holds as well for the minute clay particles, which may be even smaller than the bacteria, as it does for the bacteria themselves. If a filter has been recently scraped, the colloidal matter, or real filtering medium, must be more or less disturbed and broken up. In this case there are very probably "holes" in the filter, which must become sealed by the formation and deposition of new colloidal matter before the filter regains its original efficiency. In fact, a recently scraped filter exhibits in a limited way the same inability to strain out suspended matter, as does a filter filled with new sand which has never been used for filtering purposes.

Viewed in the light of the above theory, the height to which the loss of head may be safely carried is only relative, as the quality of the effluent depends upon several factors, which may not be even constant for the same water at different periods of the year, and which certainly vary with different classes of waters. The inconstant relation between loss of head and the quality of the effluent is a fact well known to all operators of filters, and would seem to find a rational explanation in the theory suggested above.

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CHAPTER XII

SYSTEM OF SLOW SAND FILTRATION (*Continued*)

APPARATUS FOR CONTROLLING RATE OF FLOW

Influent Controlling Devices.—The water to be applied to a filter must enter at low velocity in order not to disturb the sand

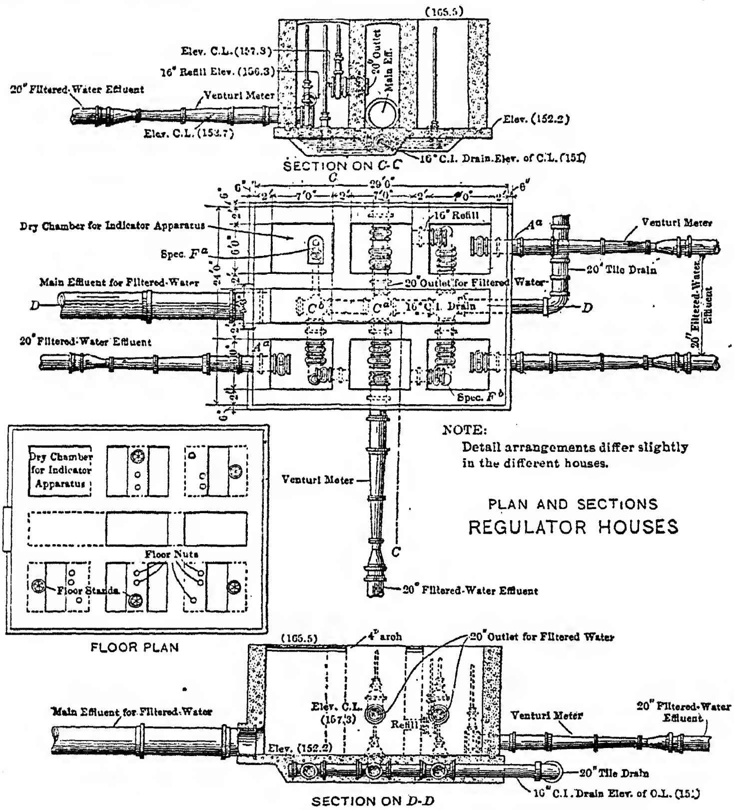


FIG. 35.—Regulator houses, Washington filtration plant.

bed. If the pressure at the point of discharge into the filter is very low, no method of control may be necessary. If the pressure is too great, however, floats on the surface of the water in the filter are used to operate a simple butterfly valve, or a balanced valve of more elaborate design, located in the supply-

pipe line. When the influent pipe discharged the water just above the surface of the sand, as was the case in some of the older plants, paving about the mouth of the pipe was utilized to prevent disturbance of the sand. In modern plants separate chambers and weirs from these chambers over which the water enters the filter proper at a low velocity, are usually provided.

In the Pittsburgh[1] slow sand filters the water is admitted through a 20-in. pipe line to an inlet chamber 6 ft. 9 in. by 9 ft. 6 in. at the inlet corner of the bed. The tops of the walls of this well are at the minimum sand surface level in the bed, and when the sand is above that level, they are carried up to the surface of the sand with bricks laid dry. The controlling valve on the influent pipe is of the disk type, the water entering it at the side and flowing out both top and bottom. The float on the stem of this valve rests on the surface of the water over the sand in the filter, and within certain limits may be set at any desired height, which height determines the depth of water over the sand.

Effluent Controllers.—In order to obtain a proper degree of purification of the water as it passes through the sand bed, it is necessary to permit it to flow at only relatively low rates. Without artificially interposing a resistance to this flow at the outlet of the filter, a clean bed of sand would pass the water at too high a rate. A gradually increasing resistance to flow is produced by the material deposited on and in the sand bed, as explained in the preceding chapter. Hence, as this latter frictional resistance increases, the artificial resistance should be proportionally decreased, if a uniform rate of discharge is to be maintained.

Numerous devices for maintaining a uniform rate of flow are in use, of which only one or two can be described. The principle involved in many of them is that of throttling the flow of water from the effluent main, so that a constant difference in head is maintained on the two sides of a fixed orifice or weir in an effluent chamber. This regulation may be effected by hand or be done automatically. In some devices there is no attempt to throttle the discharge, but in place of the stationary orifice a movable one is employed on which a constant head is kept by means of floats in an effluent chamber.

At the Albany, N. Y., slow sand filter plant, a fixed orifice, on which a constant head is maintained by hand regulation of

[1] "Pittsburgh Filtration Plant." *Eng. Record*, vol. 54, No. 24, 1906.

the effluent valve, is employed. At Hamburg, Germany, a movable weir is used, which can be adjusted by hand. Automatic regulation of the rate by means of movable weirs were provided at the Pittsburgh and Philadelphia plants. These weirs consist of telescopic tubes carried by floats. The upper end or mouth of the sliding tube can be carried at any fixed level below the surface of the water, and thus produce a constant discharge from the effluent chamber irrespective of the height of the water in the chamber, and indirectly independent of the loss of head. At the Pittsburgh plant, the float operates a valve in the effluent line, and thus controls the loss of head directly, so far as the introduction of frictional resistance to the flow of water leaving the effluent pipe could be effected. This automatic control at the Pittsburgh plant has been abandoned, as hand regulation has been found more satisfactory for the reason that the filters can be operated to a greater loss of head, and consequently with a greater yield of water between cleanings.

In some of the more modern plants the use of the Venturi meter to measure the rate of flow, has become quite common. These meters are employed at the slow sand filter plant in Washington, D. C. Regulation is effected by hand and not automatically. By utilizing the difference in pressure between that on the upstream side of the throat of the Venturi tube, and that at the throat, automatic regulation may be effected through auxiliary apparatus. This is done in several of the Venturi type of rate controllers now in use in many of the rapid sand filter plants, where the use of rate controlling apparatus of an automatic character is quite essential to the efficient operation of the plant. These rate controllers will be more fully described in a succeeding chapter.

Sand Cleaning Methods and Apparatus.—When a slow sand filter has been in service long enough to become clogged with material removed from the water during filtration, it becomes necessary to remove this material in order to maintain the quantity and quality of the effluent. Various methods are employed for cleaning the surface layers of sand, all of which are based on washing a thin surface layer of sand with clean water, and discharging the dirty wash water into the sewer. This washing may be effected by flowing water over the sand bed, and at the same time raking it with rakes to a depth of several inches, or what is more common the removal of the surface layer of dirty sand to

a depth of ½ to 1½ in., and its subsequent washing in special apparatus designed for this purpose. When the sand is to be taken out of the filter the dirty surface layer is scraped off with thin flat shovels, and deposited in piles. The dirty sand is then either wheeled out of the bed in wheel-barrows, or moved in small, tram cars to a court where it is washed. In large modern plants (Fig. 36) the sand is usually shovelled into sand ejectors and transported by water under pressure through piping to washing machines outside the filter. In some of the plants re-

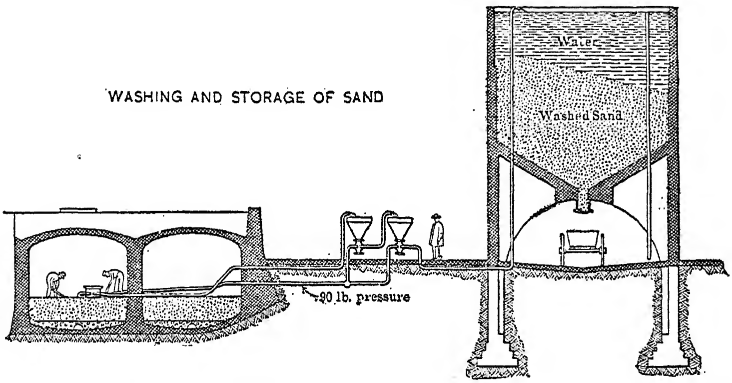


FIG. 36.—Method of washing and storing sand, Washington filter plant.

cently constructed, the dirty sand is washed by machines moving slowly over the surface of the sand bed, or by movable washers placed on the sand bed during the scraping process.

Surface Washing of the Sand Bed.—Over 20 years ago Piefke[1] described a method for cleaning slow sand filters which consisted in stirring up the surface layer of sand while a thin sheet of water was allowed to flow rapidly over the sand to a drain. The experimental results were quite satisfactory when the filter was properly arranged for carrying out the method. Within the last few years this method has been tried again with considerable success in some of the small filter plants of the New York Water Department in Long Island in the Borough of Brooklyn. It has been termed the "Brooklyn method," and is carried out after draining down the filter until only 2 or 3 in. of water remain above the sand. Drains just at the sand level are then opened at one end of the bed, and the wash water allowed to flow over the surface of the bed from an inlet at the opposite end. Boards

[1] Vierteljahresschrift für öffentliche Gesundheitspflege," 1891, p. 59.

are used to form channels about 15 ft. wide, through which the water may be directed, and to increase its velocity. While the water is passing through these channels the sand is raked in order to loosen the dirt, so that it may be washed away by the water flowing over the surface of the bed.

This method appears to be quite successful with filters clogged with certain kinds of light organic matter, such as might arise from a growth of microörganisms, for example, or from other vegetable matter; but for waters carrying any clay sediment, which penetrates the sand to some depth, it has not proven satisfactory. At the Pittsburgh slow sand filter plant, this method has been tried, but was abandoned on account of the imperfect washing which the sand received. At the Torresdale slow sand filter plant in Philadelphia, this method has also been used, but the deep penetration of the sediment produced shorter and shorter periods of service of the filters and has been discarded.

Old Methods of Sand Washing.—There have been many methods devised for cleaning dirty sand which has been removed from filter beds, ranging from simple washing with a hose stream to mechanically driven drum washers. One of the earliest devices consisted of a box with a perforated false bottom, through which the wash water was forced. The dirty sand shoveled into the box was stirred in order to assist the rising water to carry off the dirt. Washing the dirty sand as it lay on an inclined platform of plank, brick or iron with a hose stream has also been used. From 2 to 4 cu. yd. of sand were washed at a time. The dirty wash water flowed over a weir at the foot of the incline and from thence to a drain.

Mechanically driven drum washers of various designs have been very generally used in Germany. The cylinders were open at both ends; some of them were slightly conical in shape and were placed with their axes in a horizontal position. They were provided with screw blades on the inside which moved the sand from the lower to the higher end during the rotation of the drum. A stream of water was used to wash the sand as it worked its way upward through the cylinder. In some cases a narrow cylinder was used in which the sand was advanced by means of a screw in a direction opposite to a flowing stream of water. From 2.5 to 4 cu. yd. of sand could be cleaned per hour, and the quantity of water used ranged from 1,500 to 2,500 gal. per cubic yard

cleaned, according to the type of machine employed. From 2 to 4 hp. were required to rotate these drums.

Present Methods of Sand Transportation and Washing.—Although ejector (Figs. 37, 37*a* and 38) sand washers have been used

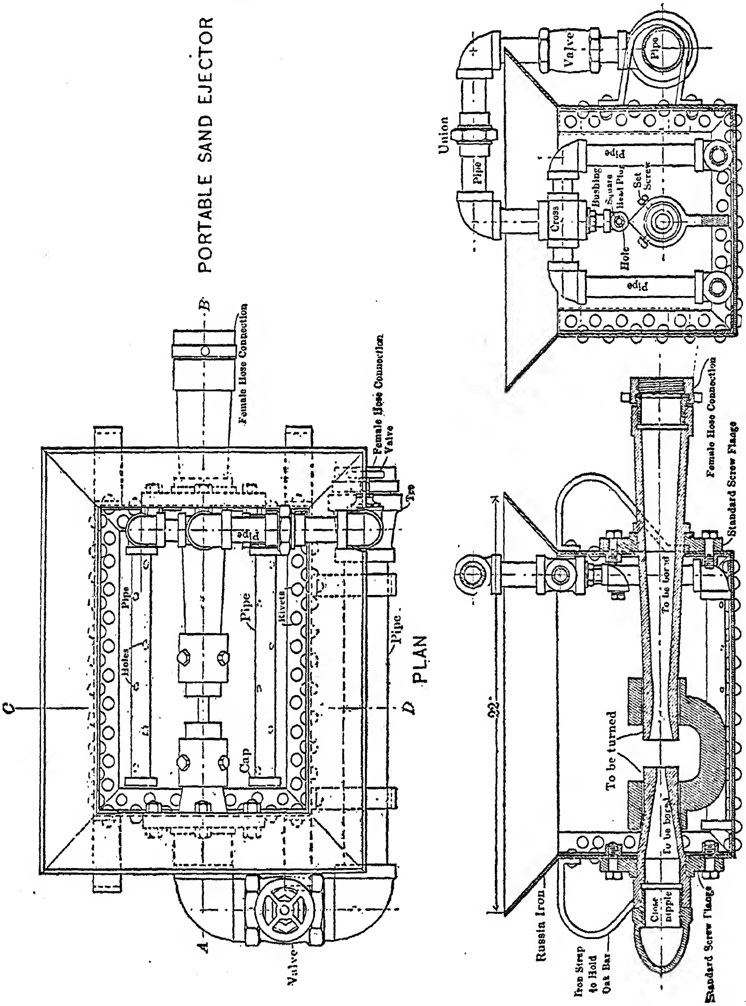
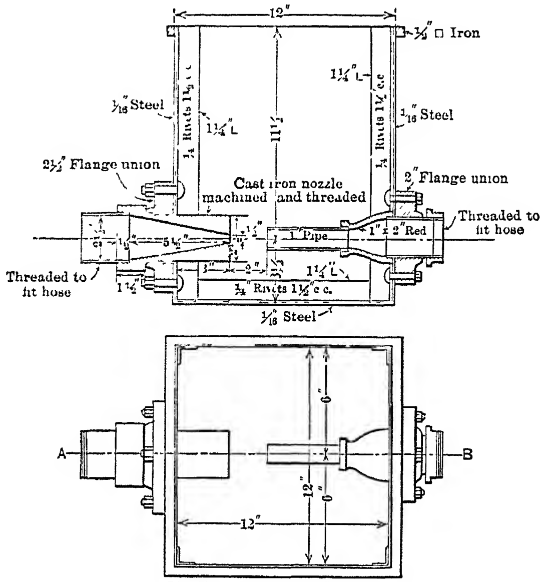


FIG. 37.—Portable sand ejector.

for many years, they have been the subject of a great deal of study within a comparatively recent period. They were first employed in England, and are now very generally used both for moving and washing sand. The hydraulics of ejectors and of the transportation of sand through pipe lines by water under pressure,

has been carefully studied by Messrs. Hazen, Hardy, Longley, Knowles and Rice.

For removing the sand from the filter, the latter is piped with high-pressure water mains (3-in. and 4-in. pipes), and with another set of pipes for carrying the sand. A portable ejector connected with the water and sand pipe lines is placed on the surface of the sand, and the piles of dirty sand which have been scraped up, are shoveled into it. The sand passes through the



PLAN AND SECTION ON AB

FIG. 37*a*.—An easily made sand ejector.

pipes with the water to ejector washers and is cleaned. It is then returned to the beds directly or is stored in courts or bins outside the filters.

The usual form of portable ejectors for removing the sand scraped from the bed is that of a cast-iron bowl with a sheet-iron extension of the sides for holding the sand. The ejector nozzle may be made of hardened tool steel, the length of the wearing part being 1¾ in., the outside diameter 1¼ in., and the bore ¾ in. The throat of the ejector is of chilled cast iron and about 7 in. long. The inner portion is a Venturi tube with the smallest diameter 1⅜ in. In a distance of 5 in. the diameter increases from 1⅜ to 2⅛ in.

At the Pittsburgh slow sand filter plant 3-in. rubber hose is used to convey the sand to the iron pipe lines, and although the

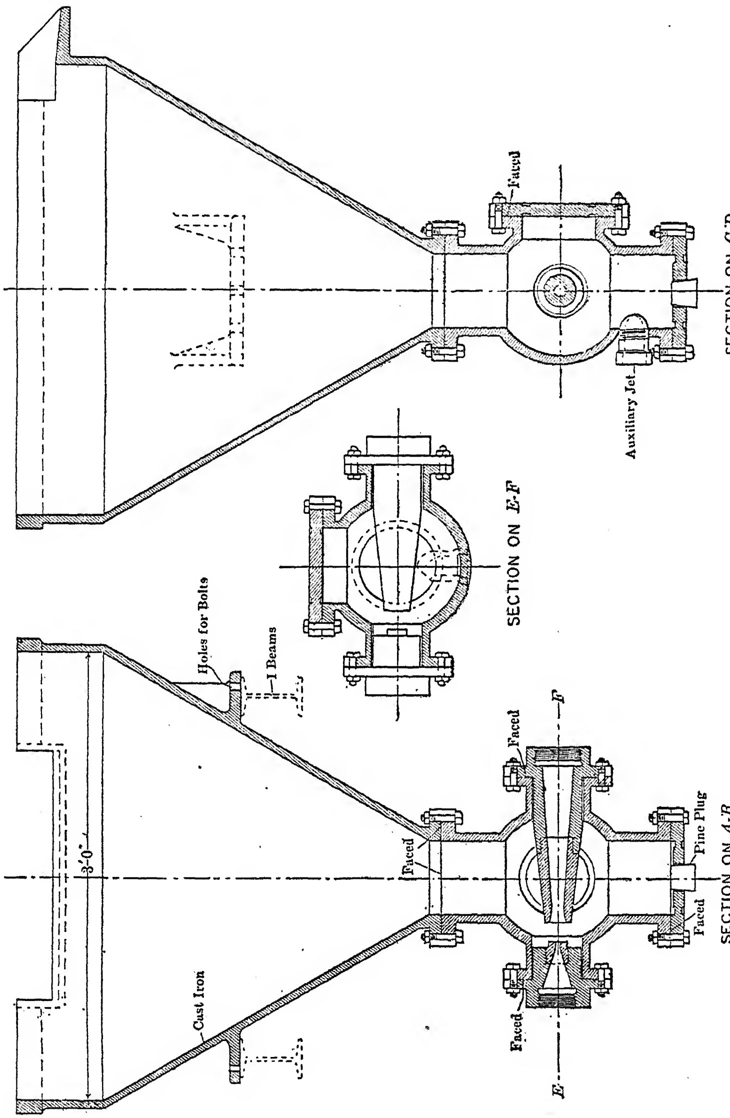


Fig. 38.—A sand washing hopper.

brass couplings and expansion rings have been found to wear, the inner rubber tube of the hose is not apparently eroded. Wrought-iron pipe 3 and 4 in. in diameter has also been much

used at this plant, and after several years' usage does not show appreciable signs of wear in the straight lengths. At the curves, however, wear on the piping has been noted, and by substituting rubber hose at the bends for the curved pipe in several instances, less trouble from erosion has resulted.

These sand ejectors will handle 8 or 9 cu. yd. of sand per hour with water pressures of 85 to 90 lb. per square inch. At the Pittsburgh filter plant, Mr. C. F. Drake states the cost during the first 7 months of 1910 for water for sand ejecting, washing, transporting and restoring to have been approximately 12 cts. per cubic yard of sand handled.

Sand-washing Machines.—In order to avoid the removal of the dirty sand from the filter and the return of the cleaned sand to it, a number of devices have been invented for washing the sand practically in place. The Nichols separator or sand washer is a portable device which may be placed upon the surface of the sand in the filter. The bed is scraped in the ordinary way, and the dirty sand is shoveled into a portable ejector and conveyed by water under pressure to the separator. The latter consists of a closed steel cylinder 42 in. in diameter and 36 in. high, and having a cone-shaped bottom. To this cone-shaped bottom are attached water-pressure and sand-discharge pipe lines. The dirty sand enters near the top of the separator and is met by a rising current of water. As the latter ascends the sand descends, and is forced out of the bottom of the separator in a clean condition. The dirty wash water is carried away at the top through hose connections to the drain pipes of the filter. The loss of sand from the separator consists of the finer material which is carried away by the rising wash water. It constitutes about 3 per cent. of the sand washed in practice. It is kept at a minimum by properly proportioning the volume of sand and of wash water, and by means of a disk and series of baffles provided within the separator. The thickness of the sand bed is not reduced where these machines are used, as the sand is immediately returned to the bed and can be spread at once if desired. Transportation of the sand for long distances to stationary washers outside of the filters is avoided by using these machines.

The separator weighs about 700 lb.; it can handle about 10 cu. yd. of sand per hour, and requires about 1,200 gal. of water for washing each cubic yard of sand. The water pressure required to operate the machine is about 65 lb. per square inch.

The Nichols separator (Fig. 39) is used at the Philadelphia slow sand filter plants where it has given very satisfactory results. The depth of sand cleaned by these machines at this plant depends upon the penetration of the sediment. and may vary from 2 in. to 10 in

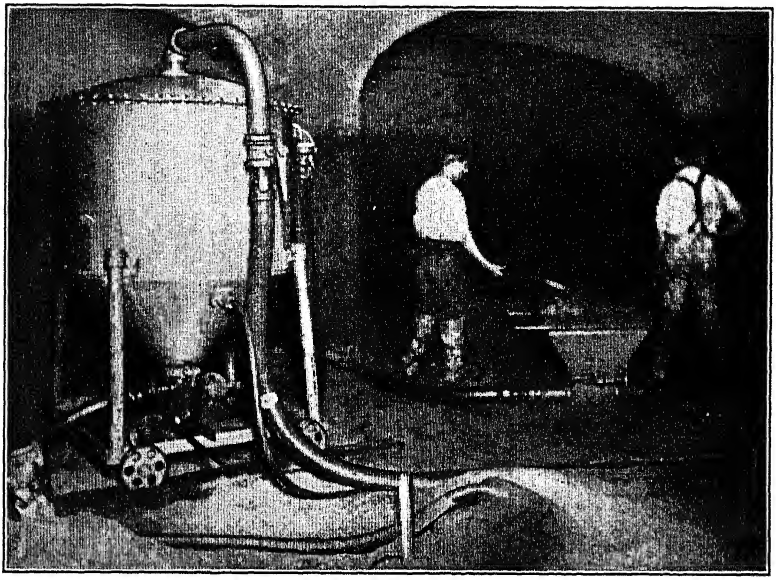


FIG. 39.—The Nichols sand-washing machine.

Blaisdell Washing Machine.[1]—This apparatus (Fig. 40) consists of an inverted box about 4 ft. square and 2 ft. deep, which is sunk in the water on the filter to the surface of the sand, and is supported from above by an iron framework on which is a platform for operating the machine. The framework rests on wheels running on rails supported by the walls on either side of the filter. Within the box is a revolving rake, which consists of a hollow axle from which project hollow teeth, and through which water is forced under a pressure of 10 to 20 lb. per square inch when the axle is revolved by an electric motor. The box may be moved vertically or horizontally by means of a motor.

In the operation of the machine, it is slowly moved along the rails by means of an electrically driven motor, causing the box with its revolving rake to slide over the surface of the sand bed.

[1] "Water Filtration and the Mechanical Washing of Filter Sand at Wilmington, Del." *Engineering-Contracting*, Aug. 26, 1914.

A suction pump draws away from the interior of the box slightly more water than is being discharged through the teeth of the revolving rake. The dirty wash water from the pump runs to waste through a gutter along the side of the filter.

The machine may be used in filters designed for washing by this method and is probably more applicable to small than to large units. It has the advantage of cleaning the sand without

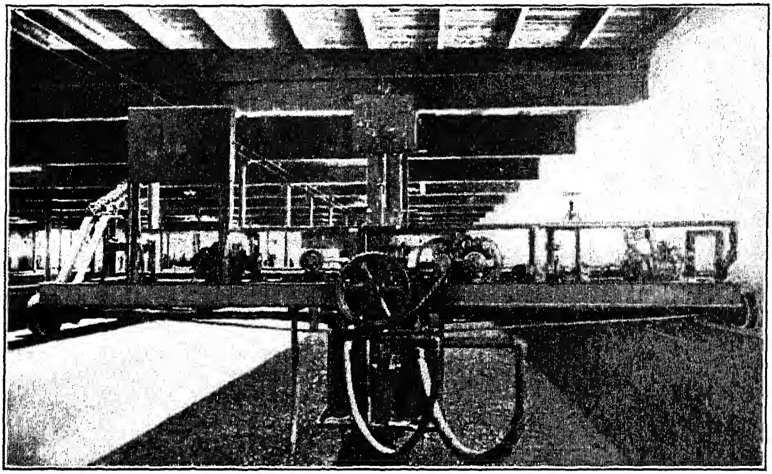


FIG. 40.—Blaisdell's sand-washing machine.

removing it and without even draining the filter, both of which operations materially decrease the net yield of filters cleaned by some of the other methods. This apparatus is used at the slow sand filter plant at Wilmington, Del., and in the preliminary filters at the Belmont plant in Philadelphia.

Methods for the Partial Cleaning of Slow Sand Filters.—The presence of organic matter in a water in the form of microscopic organisms or of mineral matter in a fine state of division frequently clogs the sand of a filter very rapidly. These adverse conditions not infrequently occur with little warning to those in charge of a plant, and necessitate resorting to rapid methods for loosening up the sand in the beds in order to maintain a supply of filtered water. The efficiency of the filtering process may have to be sacrificed to some extent in such cases, and the methods employed are regarded as only temporary expedients.

Raking.—In this method the filter is partially drained, and the sand is then lightly raked with garden rakes having teeth about

½ in. long. Four men will rake an acre bed in about 8 hr. at a cost of about $8. The filter can be refilled immediately and filtration resumed. The prolongation of the period of service by raking depends upon the character and penetration of the clogging material. The second raking is less effective than the first, and the third is usually of little value. Much deeper layers of sand have to be removed to be washed when raking has been resorted to between cleanings. The bacterial efficiency of the filter is not as much affected as one would suppose.

Spading.—Compacting of the surface of the bed may, under some conditions, proceed so far as to necessitate more effective loosening of the sand than could be obtained by raking. By going over the surface of the bed with a spade, pushing it down into the bed to a depth of 6 or 8 in., and at the same time working it backward and forward to break up the surface sand layer, a slight benefit may be obtained. It is obvious that a sand bed requiring such measures to increase its output, would produce a poorer quality of water, and that the effect of the spading would not last long. Such has been the experience at the Torresdale filter plant at Philadelphia.

Scraping and Piling.—Emergency conditions sometimes make the scraping, washing and resanding of a filter difficult, when the time of its withdrawal from service, which these processes involve, is considered. In such cases scraping off a layer of sand and piling it on the bed, until such time as it could be washed, has been tried with fairly good results. After scraping and piling, the bed may be put back into service at once. Good results have been obtained by this method at the Torresdale filter plant in Philadelphia, but it of course is only a makeshift for delaying the complete process of cleaning until a more convenient season.

Resanding Filters.—On account of the rather high cost of filter sand of good quality, it is almost invariably cleaned and returned to the beds for further use. Mr. George A. Johnson cites the only instance of which he has any knowledge where dirty filter sand is not cleaned, but is discarded. This is at Osaka, Japan, where low prices of labor (1 to 2 cts. per hour) enable clean dredged river sand to be pan-screened and prepared for the filter beds at a price of $0.65 per cubic yard. This is a price from one-fourth to one-half that for which sand may be usually obtained under favorable conditions.

In those cases where the sand is removed from the filter for

cleaning, the beds are reduced in thickness by successive cleanings. Never less than 12 in. of sand are allowed in slow sand filters, and usually the bed is quite a little thicker than this after the final scraping, and at the time the bed is refilled with sand to the original depth. When the sand is stored in courts or bins outside the filters, it is sent by means of ejectors and piping to the bed to be refilled. It is then spread by men with shovels.

It is a common practice to send the dirty sand from a filter being scraped to the ejector washers, and from them to a filter to be refilled. No storage of sand is required in such cases, if the sequence of cleanings and refillings, and the arrangement of piping makes it possible. This is the practice at the Pittsburgh slow sand filter plant. At this plant the original elaborate sand-distributing machines have been abandoned, as it has been found much cheaper to spread the sand by hand than to operate the machines.

At the Washington, D. C., filter plant, the average depth of sand replaced in refilling the filters during the period 1905-12 was 14.5 in. The yearly averages during this period ranged between 7.9 and 22 in.

In those plants where the sand is cleaned in place, as for example with the Nichols separator, only respreading of the sand by hand is required. Where the Blaisdell sand-cleaning machine is used even respreading of the sand is unnecessary. The same is also true where the "Brooklyn method" of cleaning the sand is used, although this method of washing the sand is more properly classed with those partial methods of cleaning described above.

An excellent idea of the amount of sand which it is necessary to handle in operating slow sand filters may be obtained from data taken from the 1912 report of the operation of the Washington, D. C., filter plant.

Fiscal year	Million gallons pumped to		Number of filters		Cubic yards of sand	
	Filters	Sand washers	Scraped	Raked	Removed and washed	Replaced in filters
1907–08	23,758.24	76.98	126.67	86	22,275	21,393
1908–09	22,435.16	119.16	137.80	36	24,683	23,487
1909–10	21,605.44	91.78	87.20	51	15,505	16,876
1910–11	22,039.95	79.92	91.00	71	14,941	13,218
1911–12	22,668.40	84.86	104.40	83	14,820	16,095

COST OF CONSTRUCTION OF SLOW SAND FILTERS

The cost of constructing slow sand filters varies greatly. It depends upon local conditions to such an extent that comparative figures may be misleading, unless given in considerable detail. It is not always possible to separate the costs of the various parts of a plant, owing to the manner in which the original cost accounts were kept. Changes in the cost of labor and material naturally influence the actual cost of plants which have been built at different times during the past 20 or 25 years.

The cost per acre for slow sand filters will be found to range usually from \$45,000 to \$75,000. Covered filters will generally cost about 50 per cent. more than open filters. An average cost for slow sand filters will be found to be about \$60,000 per acre. In exceptional cases the above figures may be much less or much more than those stated above. The filter at Lawerence, Mass., built in 1893, cost about \$27,000 per acre of filtering surface. Mr. G. A. Johnson[1] states that an uncovered filter built in 1903 is Osaka, Japan cost \$31,000 per acre. At the other extreme in costs may be cited some of the Philadelphia slow sand filter plants. The Torresdale covered filters cost about \$145,000 per acre, and the Belmont covered filters approximately \$187,000 per acre, which latter, however, includes the cost of a clear-water reservoir of 16,500,000 gal. capacity.

Detailed costs of the Albany, N. Y., Washington, D. C., and Pittsburgh, Pa., slow sand filter plants, as stated by their designers and as tabulated by Mr. G. A. Johnson in the above-mentioned paper, are as follows:

Detailed Costs of Slow Sand Filtration Plants

Albany, N. Y.

Daily capacity 15,000,000 gal.

Land	\$8,290
Pumping station and intake, complete	49,745
Filters, eight beds, each 0.7 acre in area; sedimentation basin, capacity 37,000,000 gal.; and filtered-water reservoir, complete	324,217
Conduit and connections with Quackenbush Street Pumping Station	86,638
Engineering and contingencies	31,000
Total approximate cost of works	\$499,890

[1] G. A. Johnson: Water Supply Paper, U. S. Geological Survey, No. 315, 1913.

Cost of filters per acre	$45,600
Cost of uncovered sedimentation basin per million gallons capacity	4,100
Cost of filtered water reservoir per million gallons capacity	15,000
Cost of filters per million gallons gross daily capacity	15,200
Cost of works per million gallons daily capacity	33,320

Washington, D. C.
Daily capacity 75,000,000 gal.

Land	$619,900
Pumping station, including intake, Venturi meter, electric generating plant, stack, etc., complete	183,600
Twenty-nine covered filters, each 1 acre in area, complete	2,197,000
Filtered-water reservoir, capacity 14,200,000 gal., including gate house and regulating apparatus, complete	150,000
Lower gate house and pipe line	24,300
Engineering and clerical work	181,500
Total cost of works	$3,356,300
Total cost excluding land	$2,736,400
Cost of filters per acre	75,700
Cost of filtered water reservoir per million gallons capacity	10,600
Cost of filters per million gallons gross daily capacity	25,250
Cost of plant per million gallons daily capacity	44,750

Pittsburgh, Pa.
Daily capacity 120,000,000 gal.

River crossing and connections	$298,589
Low-lift pumping station	357,513
Low-lift pumping machinery, boilers, etc.	284,169
River-well and intake	207,673
Brilliant pumping station (additional machinery)	221,472
Pipe lines to Highland reservoir	555,250
Filters (46 1-acre units, covered),[1] and open sedimentation basins	3,586,245
Filtered-water reservoir, 45,000,000 gal. capacity	460,060
Total cost of works	$5,970,971
Cost of filters (including settling basins) per acre	78,000
Cost of filtered-water reservoir per million gallons capacity	10,200
Cost of filters (including settling basins) per million gallons daily capacity	26,000
Cost of plant per million gallons daily capacity	42,600

NOTE.—Above costs compiled from the award of the arbitrators to the contractors in July, 1910, and does not include engineering and a small portion of day work done by the City of Pittsburgh.

[1] Ten more acres of filters have been built since the first 46 filters were built, but have never been put in service.

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5. *Trans.* Amer. Soc. C. E., vol. 43, 1900; and vol. 57, 1906; pp. 294 and 307, respectively.
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   (*a*) "Pittsburgh Filtration Plant," vol. 54, No. 24, 1906.
   (*b*) "Lower Roxborough Filters," Dec. 8, 1900 and Jan. 3, 1903.
   (*c*) "Upper Roxborough Filters," Apr. 13, 1901, Mar. 28, and Apr. 11, 1903.
   (*d*) "Belmont Filters," Sept. 12, 1903.
   (*e*) "The Bernhart Sand Filters, Reading, Pa.," vol. 60, Nov. 13, 1909.
   (*f*) "Toronto Water-purification Works," vol. 60, Sept. 18, 1909.
   (*g*) "Purifying St. Lawrence River Water at Ogdensburg, N. Y.," vol. 64, p. 642.
   (*h*) "Notes on the Wilmington Filters," vol. 63, p. 310, 1911.
   (*i*) "The Didelon Filter-bed Regulator," vol. 57, Apr. 11, 1908.
   (*j*) J. H. Gregory: "Slow Sand and Mechanical Filter Costs," vol. 69, June 20, 1914.
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CHAPTER XIII

EFFICIENCY AND COST OF OPERATION OF SLOW SAND FILTERS

EFFICIENCY OF SLOW SAND FILTERS

The physical properties and sanitary quality of waters after passage through slow sand filters depend upon the character of the applied water, and upon the skill with which the plant is operated. When finely divided suspended matter in the raw water is applied to the filters for too long a period, an effluent of unsatisfactory appearance is sure to result. Usually such conditions also produce an effluent of high bacterial content, which may be unsafe, or at least undesirable to supply for consumption. Difficulties with microörganisms, which produce odors that are carried through the sand with the filtered water, are also sometimes encountered.

In operating slow sand filters the limitations of their efficient action must be thoroughly appreciated. Only by the most careful observation of each filtering unit during its period of service, and by a comprehensive system of records showing the performance of the plant as a whole, can it be expected to supply a water of uniformly high quality. Even with the most conscientious attention, plants which are incorrectly designed or inadequately provided with methods for supplying a water properly prepared for the filters may become difficult to operate, and may produce effluents of an unsatisfactory character.

Periods of Service and Yield.—The length of time which slow sand filters may be operated efficiently varies greatly. The yield of the filters as regards quantity, as well as quality, depends upon the frequency of cleaning and upon its thoroughness. These points are well illustrated in the following table taken from a paper[1] of Messrs. Francis D. West and Joseph S. V. Siddons on the operation of the Torresdale plant at Philadelphia.

[1] *Jour. New England Water-works Assn.*, vol. 27, Sept., 1913, p. 358.

RESULTS OF CLEANING SLOW SAND FILTERS BY VARIOUS METHODS AT TORRESDALE FILTRATION PLANT, PHILADELPHIA, PA.

No of filters cleaned	Yield per filter			Yield per acre, millions of gallons	Rate per acre	Period of service, days,		
	Ave	Max	Min.	Average	Average	Ave	Max	Min
					First periods of service			
65	40	73	21	53	1 6	43	77	11
					Cleaning by Brookyln method			
609	19	67	4	25	1 8	14	49	6
					Cleaning by scraping and piling			
530	35	69	17	47	2 4	19	39	10 (*a*)
122	40	142	3	53	4 0	14	57	2 (*b*)
					Cleaning by ejection from filter			
282	39	80	19	52	2 6	20	45	10 (*a*)
235	116	379	10	155	4.5	34	105	4 (*b*)
					Cleaning by raking			
,012	85	182	3	113	5 0	24	50	2 (1st)*
408	50	79	16	67	4.3	16	24	7 (2d)*
62	52	..	39	...	....	13	....	.. (3d)*
					Cleaning by Nichols' sand separator			
1,029	141	569	49	188	4.4	42	148	11
					General averages			
1,486	30	80	4	40	2.1	18	77	6 (*a*)
2,958	96	569	3	128	4.4	29	148	2 (*b*)

NOTES —(*a*) Before pre-filters were placed in service; (*b*) after pre-filters were placed in service. Asterisk refers to results obtained after "first raking," "second raking" and "third raking," respectively. Data summarized through 1912, except the maximum, minimum and rate of filtration data under "rakings," and a method of spading in 1912 are omitted from the averages.

The authors conclude that:

"A plant constructed as Torresdale, without any sedimentation basin, is utterly unable to cope for any prolonged period with water having a turbidity of over 100, that is, with the slow sand filters operating at a 6,000,000-gal. rate. When such a condition is reached, the pre-filters fail to do their proportion of the work, and the final filters choke badly, allowing fine silt to pass through them. This choking necessitates cleaning for 24 hr. a day with 55 to 58 filters doing the work of 65, and depending on calcium hypochlorite to reduce the number of bacteria and destroy the pathogens. Fortunately the periods of turbid water occur but seldom and are of short duration."

In spite of these occasional adverse conditions the average bacterial efficiency of this plant is over 99.5 per cent.

Removal of Sediment and Bacteria.—The number of bacteria removed by well-operated slow sand filters usually ranges between 98 and 99 per cent. of the number originally present in the raw water. If preliminary treatment of the raw water, by some of

the various methods previously described, is employed, the effluent of the filters is likely to contain less than 100 bacteria per cubic centimeter for a greater part of the time. The turbidity of the applied water greatly influences the number of bacteria found in the effluent. This is well shown by the following tables taken from Messrs. West's and Siddons' paper on the operation of the Torresdale filters which was referred to in a preceding paragraph.

TABLE SHOWING RANGE IN TURBIDITY, 1912

Turbidity	Delaware River	Effluent of roughing filters	Turbidity	Filtered-water basin
Parts per million	Days	Days	Parts per million	Days
0–10	154	297	0–1	339
11–25	146	41	2–10	18
26–50	38	8	11–25	7
51–100	9	6	25–51	2
101–250	9	6	Over 51	0
251–500	3	6	......	...
501–750	4	2	.	....
751–1,050	3	0	. .	. . .
Over 1,050	0		.	

TABLE SHOWING RANGE IN BACTERIA, 1912

Bacteria	Delaware river	Effluent of roughing filters	Bacteria	Filtered water basin
Per c.c.	Days	Days	Per c.c.	Days
0–100	0	....	0–5	85
101–200	0	...	6–10	101
201–300	0	1	11–25	107
301–400	0	3	26–50	34
401–500	0	6	51–100	15
501–750	1	19	101–200	12
751–1,000	6	55	201–500	4
1,001–1,500	16	44	501–1,000	5
1,501–2,000	36	42	1,001–2,100	2
2,001–2,500	27	30	Over 2,100	0
2,501–5,000	92	93	....	....
5,001–10,000	86	36	....	....
10,001–25,000	67	28	....	....
25,001–50,000	22	6	....	....
50,001–100,000	10	2	....	....
Over 100,000	3	0	....	....

The results obtained at the Washington filtration plant also show the difficulties encountered in the operation of slow sand filter plants when treating turbid waters. A series of large reservoirs, through which the Potomac River water passes before it reaches the filter plant, afford ample periods of sedimentation and storage, and make it possible to keep out of the system quite a large volume of the muddiest water which would otherwise have to be filtered. During the fiscal year July 1, 1911, to June 30, 1912, the inlet gates from the river at Great Falls were closed 21.14 per cent. of the time, and excluded 30.28 per cent. of the total suspended matter. Treatment with alum and subsequent settlement was also practised in order to reduce the turbidity of the water applied to the filters. The average turbidity in the filtered water for the month of January, 1912, was 1 part per million, although a maximum of 4 parts per million were present at times during the month. Before coagulation of the sediment in the water with alum was attempted, the maximum turbidity of the filtered water sometimes reached 20 parts per million; and the yearly averages for the years 1906–10 were between 1 and 2 parts per million.

The bacterial removals by this plant are exceptionally high, and are best shown by the following table of yearly averages:

Average Number of Bacteria per Cubic Centimeter in Settling Reservoirs and in Filtered-water Reservior[1]

Year	Dalecarlia reservoir		Georgetown reservoir outlet	McMillan Park reservoir outlet	Filtered-water reservoir
	Inlet	Outlet			
1906–07	4,850	1,940	1,680	635	31
1907–08	6,300	2,700	2,940	1,250	55
1908–09	3,130	1,950	950	390	21
1909–10	14,300	13,850	10,850	6,820	143
1910–11	4,820	3,370	2,080	1,390	38
1911–12	8,200	6,000	2,600	1,100	35

The following table taken from the same report, indicating the percentage of positive tests for the Bacillus coli found in testing 1 c.c. of water from samples taken from the various parts of the system of reservoirs and from the effluent from the filter plant is also of interest:

[1] Report upon the Maintenance and Repair of the Washington Aqueduct, District of Columbia, and Filtration Plant, 1912.

PERCENTAGE OF POSITIVE TESTS FOR THE BACILLUS COLI FOUND IN 1 C.C. OF WATER FROM SAMPLES FROM SETTLING RESERVOIRS AND FILTER PLANTS

Year	Dalecarlia reservoir		Georgetown reservoir outlet	McMillan Park reservoir outlet (applied water)	Filtered-water reservoir
	Inlet	Outlet			
1905–06	19.4	23 2	14 9	8 3	1 8
1906–07	43.6	29 2	29 8	13 0	2 1
1907–08	31 3	12 3	22 1	9 4	0 3
1908–09	20 3	15 0	8 5	7 1	0.0
1909–10	26 9	24 0	19.8	10.4	0.8
1910–11	16.8	11 5	3 6	2.2	0.0
1911–12	43 5	34 3	21.1	10 5	0.8

At the Wilmington, Del., slow sand filter plant, the raw water does not carry so much sediment but is very high in bacteria. The following table condensed from the annual report for 1911–12 of the Water Commissioners of Wilmington, shows the efficiency of this plant:

Averages for 1911–12	Raw water	Pre-filtered water	Applied water	Combined effluents
Turbidity, parts per million.......	68	36	30	2
Bacteria per cubic centimeter......	49,533	29,816	21,795	472
Percentage efficiency..	.. ..	39.81	26.91	99.05
Percentage positive B. coli tests in 1 c.c..	99.3	94 40	89.90	21.90

The wide variation in the results obtained at different purification plants indicates great differences in the amount of pollution with which the plants have to contend, and also the effect on the water of various methods of treatment prior to its final passage through the filters.

The maximum, minimum and average reductions of both the turbidity and the bacteria by each of the slow sand filter plants of Philadelphia are well shown in the following table taken from the Report of the Department of Public Works, Bureau of Water, for the year 1912. The comparison is made between the water applied to the filters and their effluents.

	Source of supply				
	Schuylkill River			Delaware River	
	Lower Rox-borough	Upper Rox-borough,	Belmont	Queen Lane	Torresdale
	per cent.	per cent	per cent	per cent	per cent
Turbidity:					
Average reduction. . . .	98.85	99.15	98.95	91 25	93.74
Maximum reduction.	100 00	100.00	100.00	97.50	100 00
Minimum reduction. . . .	97.37	90 00	95.45	. . . .	88 33
Bacteria:					
Average reduction.	99.49	99.63	98.96	96.42	99.03
Maximum reduction. . . .	99 97	99 96	99.95	99.69	99.90
Minimum reduction . .	96.91	97.50	95.05	75 00	96.90

NOTE.—The report does not state whether disinfectants were used or not, but if they were used the bacterial reductions are probably the result of filtration and disinfection combined.

COST OF OPERATION OF SLOW SAND FILTERS

The principal item of cost in the operation of slow sand filters is that for cleaning and moving the sand. The handling of large quantities of sand obviously involves considerable labor, and even where modern methods are used for ejecting, cleaning and restoring the sand to the filter beds, the cost of these operations must of necessity form no inconsiderable part of the total cost. If preliminary filters are employed to prepare the water for the slow sand filters, the cost of operating them is properly chargeable to the process as a whole; and if chemical coagulants or disinfecting compounds are used they must also be regarded as a charge to be included in the total cost. In open filters the cost of removing ice may be quite high, depending on the quantity which must be handled. The cost of pumping the water to the filters is sometimes included in the cost of operation of filter plants, but can be usually regarded as not chargeable to the process.

In some of the older plants in the United States the cost of scraping, washing and restoring sand has been in the neighborhood of $1.50 per cubic yard. Mr. George W. Fuller[1] some years ago tabulated some costs of handling sand, as follows: Lawrence,

[1] *Trans.* Am. Soc. C. E., vol. 46, 1901, p. 336.

Mass., $1.70 per cubic yard; Mount Vernon, N. Y., $1.51 per cubic yard; and Albany, N. Y., $1.38 per cubic yard. Very much lower costs have been obtained in some of the larger and more recently built plants. This is well shown in the following tables taken from the annual report of 1911–12 on the operation of the Washington, D. C., filter plant.

Average Cost of Labor for Sand Handling

Fiscal years	Scraping	Ejecting	Washing	Smoothing	Raking	Resanding	Total
	A. Per million gallons pumped to filters						
1905–06	$0 06	$0 29	$0 02	$0 06	.....	$0 04	$0 47
1906–07	0 07	0 20	0 05	0 02	. ..	0 24	0 58
1907–08	0 09	0 14	0 03	0 01	$0 02	0 13	0 42
1908–09	0.07	0 15	0.03	0 01	0 01	0 14	0.41
1909–10	0 05	0 10	0.01	0 01	0 01	0 08	0 27
1910–11	0 06	0 08	0 01	0 01	0.02	0 03	0 21
1911–12	0.06	0 08	0 00	0 01	0 02	0 03	0.20
	B Per cubic yard of sand						
1905–06	$0.07	$0.35	$0.04	$0.07	.....	$0.14	$0.67
1906–07	0 06	0.19	0 03	0.02	. .	0.17	0.47
1907–08	0 09	0.15	0 03	0.01	.....	0 14	0.42
1908–09	0 06	0.14	0 03	0 01	.....	0 13	0.37
1909–10	0 07	0 14	0 02	0.02	.....	0.10	0 35
1910–11	0.09	0.12	0.01	0.01	.....	0.05	0 28
1911–12	0.09	0.12	0 00	0.01	.. .	0 05	0.27

The cost of handling sand at the slow sand filter plant at Pittsburgh, Pa., is somewhat higher than at Washington, D. C., as shown by the following table which has been condensed from a large table of unit costs given in the Report of the Bureau of Water for the year 1912.

1912	Per acre	Per cubic yard	Per million gallons of water filtered
Cost of scraping sand.............	$25.41	$0.178	$0.326
Cost of ejecting sand.	....	0.245	0.440
Cost of washing and transporting sand.	......	0.120	0.216
Cost of restoring sand.	....	0.073	0.128
Cost of raking sand...............	8.66	....	0.078
Total cost.......................	....	$0.616	$1.188

Note.—Cost of common labor is $2.10 per day of 8 hr.

As 33,467,000,000 gal. of water were filtered at this plant during 1912 at a total cost of $114,238.16, the cost per million gallons filtered was $3.413. These figures include both maintenance and operation.

At Wilmington, Del., the total cost for the year 1911–12 for operating the slow sand filter plant was $1.821 per million gallons, divided as follows:

Preliminary filtration.	$0.375
Slow sand filtration	0 916
Laboratory charges.	0.530
Total..	$1.821

These are gross operating costs and do not include interest or depreciation on the plant investment.

The cost of operating five different slow sand filter plants in Philadelphia, Pa., during the year 1912, was as follows:

Cost per million gallons	Upper Roxborough	Lower Roxborough	Belmont	Torresdale	Queen Lane	Average
Pre-filters......	.....	$2.10	$0.43	$0.24	$0.22	$0 30
Final filters....	$3.59	3.52	3.45	1.67	2.31	2.13
Pumping station..	4.40	. .	....	.	.. .	0.18
Total cost.....	$7.99	$5.62	$3.88	$1.91	$2.53	$2.61

Some very valuable detailed costs of the operation of the slow sand filter plant at Washington, D. C., are given in the report on the operation of this plant for the fiscal year 1911–12.

Cost per Million Gallons Filtered

Fiscal years	Office and laboratory	Pumping station	Filter operations: Sand handling	Filter operations: Incidentals and repairs	Preliminary treatment	Care of buildings and grounds	Construction (new work)	Main office	Experimental filters	Total
				A. Labor						
1909–10	$0.84	$0 70	$0.27	$0.23	. . . .	$0 36	$0.02	$0.14	..	$2.56
1910–11	0.78	0.65	0.21	0 07	......	0 23	0 29	0.05	. .	2.28
1911–12	0.79	0.62	0 20	0.07	$0 05	0 41	0.09	0.09	.	2 32
				B. Material						
1909–10	$0.13	$0.69	$0.02	$0.05	......	$0.17	$0.16	$0.02	. ...	$1.24
1910–11	0.08	0.69	0.05	0.08	......	0.17	0.36	0.01	.....	1.44
1911–12	0 17	0.62	0.04	0.03	$0.48	0.20	0.15	0.00	.....	1.69
				C. Totals						
1909–10	$0.97	$1 39	$0.29	$0 28	... .	$0.53	$0.18	$0 16	. ..	$3.80
1910–11	0 86	1.34	0 26	0.15	......	0.40	0.65	0.06	.....	3.72
1911–12	0.96	1.24	0.24	0.10	$0 53	0.61	0.24	0.09	.....	4 01

If the cost of pumping is deducted from the above total costs, it will be seen that the remaining costs are \$2.41, \$2.38 and \$2.77 per million gallons, respectively, for the fiscal years tabulated.

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CHAPTER XIV

RAPID SAND FILTRATION

The rapid filtration of water through beds of sand, as distinguished from slow filtration, is characterized not alone by the difference in the rates of flow of the water through the bed, but by several other features which naturally differentiate the two general methods of purification. A high rate of filtration necessitates the rapid formation on the sand grains of a colloidal coating, so that the latter will entrap and prevent minute particles of suspended matter from passing through the sand bed.

This is accomplished by the use of chemical coagulants. Since the rate of flow is usually from 100,000,000 to 125,000,000 gal. per acre daily, it is obvious that relatively small areas are required for filtration as compared with those needed for slow sand filters. The sand also naturally becomes dirty very quickly, and rapid and effective methods for cleaning the sand beds are required. To meet these conditions a type of filter has been evolved which bears little resemblance to the slow sand filter.

Preparatory treatment of water for rapid sand filters requires special basins for obtaining the effect of the chemicals applied to the water. These coagulation basins, as they are called, are now considered essential features of this process of purification. The latter phase of water purification has been discussed in a previous chapter, and will not be considered in that which follows except as it may bear upon the construction and operation of the filters.

Development of Rapid Sand Filters.—The clarification of water containing suspended matter offers difficulties which the pioneers in this field did not fully appreciate, and which were overcome only by patient investigation and experiment. The skill and inventive genius displayed in the construction of the first rapid sand filters have not always been appreciated. The later scientific investigations utilized the earlier experiences of the builders of "mechanical filters," and placed this method of water purification upon a rational and scientific basis.

Mechanical filters were first developed in the United States in connection with the paper industries, in which it was necessary

to remove the coarser particles of suspended matter in the water used in the manufacturing process in order not to injure the texture of the paper. These filters consisted of cylindrical tanks containing a layer of sand, and some form of strainer system at the bottom of the tank, through which the filtered water could escape. They were incapable of removing to any great degree the finely divided clay or the bacteria, and their adaptability to the purification of municipal water supplies was given little consideration. The employment of a chemical coagulant to prepare the water for filtration produced such good results, however, that many mechanical improvements were made in filters of this type. These improvements consisted chiefly of better strainer systems, which produced a more uniform draft on all parts of the sand bed when filtering, and which also enabled a more even distribution of wash water, when the filter was being washed by forcing water up through the sand; of ingenious methods for stirring the sand when washing the bed with water; of convenient arrangements of piping, valves and drainage gutters; and of provisions for treating the water with chemical coagulants in separate tanks, or compartments of the filter tank prior to filtration.

The first rapid sand filter plants constructed for purifying municipal water supplies were either moderately successful, as measured by present ideas of efficient filtration, or dismal failures. The more successful plants handled waters which did not carry much suspended matter, except possibly for very short periods, while the unsuccessful plants attempted the treatment of waters which were constantly turbid and which were heavily charged with suspended matter for a considerable portion of the year. The method of operating these plants was entirely empirical, which led naturally to many failures, some of which were extremely costly to the promoters of this type of plant. In spite of the reverses met in the early development of mechanical filters, the successes achieved were sufficient to attract the attention of investigators, who attacked the problems involved from a scientific standpoint.

Experimental Work.—The experiments of Mr. Edmund B. Weston at Providence, R. I., were undertaken in 1893–94. They demonstrated the possibility of obtaining fairly satisfactory results in purifying the water of the Pawtuxet River by means of a small mechanical filter. This river is a small stream

in New England, the water of which is practically free from turbidity, but is more or less colored from vegetable matter. From 70 to 90 per cent. of the coloring matter was removed by the filter, and on an average about 98.5 per cent. of the bacteria. The experiments were not, however, entirely conclusive, and after a period of investigation covering about 10 months were discontinued.

The river waters of New England carry as a rule but little sediment, and while the above experiments had demonstrated the ability of this type of filter to successfully purify such waters, its usefulness for clarifying and purifying very turbid waters, so that they might be made suitable for drinking purposes, had not been investigated. The turbid water of the Ohio River offered an excellent opportunity to test the efficiency of mechanical filters, and the importance of establishing their real worth was recognized by Mr. Charles Hermany, chief engineer of the Louisville Water Co. of Louisville, Ky. By coöperating with the manufacturers of various patented filters, Mr. Hermany established an experimental station which was located at the river pumping station of the Louisville Water Co. on the bank of the Ohio River. A laboratory was also built and equipped for use in testing the water before and after filtration through the various filters which had been installed.

The experimental work was placed in charge of Mr. George W. Fuller, who was assisted by a staff of engineers, chemists and bacteriologists. The filters were operated by representatives of the manufacturers. The experimental work extended over a period of about 2 years between the latter part of the year 1895 and the middle of the year 1897. This extensive investigation is covered in a report by Mr. George W. Fuller.[1]

The conclusions reached as a result of this work have been of much value in the development of this type of filter, and established the fundamental principles underlying the art. In general it was shown that turbid waters could be successfully clarified and purified by this process, but that preliminary subsidence with the assistance of coagulants was essential, and that longer periods for settlement than had previously been employed were necessary in order to obtain efficient results from the filters. The efficiency of the experimental filters in removing bacteria

[1] Report on the Investigations into the Purification of the Ohio River Water at Louisville, Ky., 1898.

ranged from 97.5 to 98.5 per cent., when operating under proper conditions.

In order to determine whether slow sand filters or any modification of this type of filter could be used to clarify and purify the Ohio River water, there were instituted a series of experiments early in 1897 at Cincinnati, Ohio. This work was undertaken by the City of Cincinnati through its Commissioners of Waterworks, who were about to build a new water-works system, including a purification plant. The work was carried on under the direction of Mr. Gustave Bouscaren, chief engineer for the Commission, and under the immediate charge of Mr. George W. Fuller, assisted by a staff of experts.

In this experimental work ample sedimentation periods were obtained in large settling tanks. The settled water was applied to slow sand filters constructed with sand beds of different depths. The results showed that satisfactory effluents could not be obtained by even prolonged periods of sedimentation, and subsequent filtration through a bed of sand even 5 ft. in depth. Modification of this process, whereby preliminary coagulation of the settled water preceded filtration, indicated that much better results could be obtained. In consequence a rapid sand filter for experimental use was installed in order to obtain a comparison between water treated by the two different processes. The main conclusion reached in this work was that both the modified slow sand filter process and the rapid sand filter process were able to produce a water of satisfactory quality, but that the method with rapid sand filters was the more economical. The employment of rapid sand filters preceded by plain sedimentation and followed by an adequate period of settlement after coagulation with chemicals was, therefore, recommended as being able to produce a filtered water which would be completely clarified, and which would contain less than 100 bacteria per cubic centimeter on an average.

In 1897 Mr. Allen Hazen conducted tests on some rapid sand filters at Lorain, Ohio. These filters treated the water of Lake Erie which carries little sediment, but which at this particular locality contained a large number of bacteria. The bacterial efficiency of the filters averaged a removal of 96.4 per cent.; the clarification of the water was hardly a question at issue in these experiments since the water was practically clear before filtration.

Mr. Hazen carried out with experimental filters at Pittsburgh,

Pa., in 1898, another series of experiments on Allegheny River water, which latter may be classed as a normally turbid water, although not as turbid as the water of the Ohio River. The results of this work did not differ materially from those obtained at Louisville, Ky.

In 1899, Col. A. M. Miller, U. S. A., conducted a series of tests on the Potomac River water at Washington, D. C. This river water is at times very turbid, although large reservoirs in the supply system enable the water to be subjected to long periods of settlement. Rapid sand filters were recommended for purifying this supply, but were not adopted. The experiments showed that this type of filter would be able to produce satisfactory results, although it was also thought that a slow sand filter system assisted by preliminary coagulation would be able to produce the desired quality of water. It is of interest to note that after the Washington slow sand filter plant had been operated a number of years, the preliminary treatment with coagulants has been adopted, and has materially improved the quality of water delivered by the plant.

An interesting series of experiments were conducted by Mr. R. S. Weston at New Orleans in 1901 on the Mississippi River water. Both slow sand and rapid sand filters were tested in connection with preliminary sedimentation. The excessive quantities of sediment carried by the river water required too long periods of settlement to economically prepare it for slow sand filters, although satisfactory effluents could be obtained from the latter type of filter if preliminary settlement was sufficiently prolonged and supplementary subsidence with coagulants was practised. The most economical process was that which included plain sedimentation, followed by settlement with chemical coagulants, and final filtration through rapid sand filters.

The general conclusions reached in these early experiments in different parts of the United States may be briefly summarized as follows:

First.—The physical, chemical and bacteriological characteristics of any given water should be well understood if it is to be successfully purified by this process.

Second.—Those waters carrying much suspended sediment require preliminary settlement to remove the coarser particles, followed by a shorter period of sedimentation with chemicals in order to coagulate and precipitate the finer sediment.

Third.—Rapid filtration through a sand bed is satisfactory only where the sediment in the water is properly coagulated, and where the quantity remaining in the water and which is applied to the filter bed is not too great.

Fourth.—The rate of filtration should be carefully controlled, and rapid fluctuations in the rate made impossible.

Fifth.—The amount of chemical coagulant applied should be properly adjusted to the character of the water to be purified, and its application should be uniform.

Sixth.—Chemical and bacteriological tests should govern the operation of rapid sand filters, and intelligent supervision of their operation should be regarded as essential if a properly purified water is to be produced.

The value of the investigations carried on between the years 1893 and 1901 in the practical development of the rapid system of sand filtration can hardly be overestimated. They not only defined the general principles involved, but they pointed out the practical methods by which the process might be developed in conformity with those principles. Reports covering all the data obtained in these various investigations have been published, and reference to them should be made if a detailed study of this experimental work is desired.

Early Types of Mechanical Filters.—There are two general classes into which rapid sand filters may be divided, viz., pressure filters and gravity filters (Figs. 41 and 42). Pressure filters were among the earliest filters constructed, but have never reached a state of development equal to that of gravity filters.

Pressure Filters.—The filters of this class are closed cylinders of steel or iron, in which is placed a bed of sand, and through which the water is forced under pressure. Sometimes other filtering materials are used beside sand, such as charcoal or coke, but sand is more commonly employed. A compartment of the closed cylinder is often used as a small coagulation chamber from which the treated water flows to the filtering compartment. The latter is provided with a strainer system of some form through which the filtered water escapes after passing through the filter bed. The strainer system also acts as a distributor for the wash water during the cleaning of the sand bed. No method of agitation of the sand prior to or during the washing process was employed in the earliest pressure filters, but later on compressed air began to be used, and is still the more common method of assisting the wash water in cleansing the filter bed.

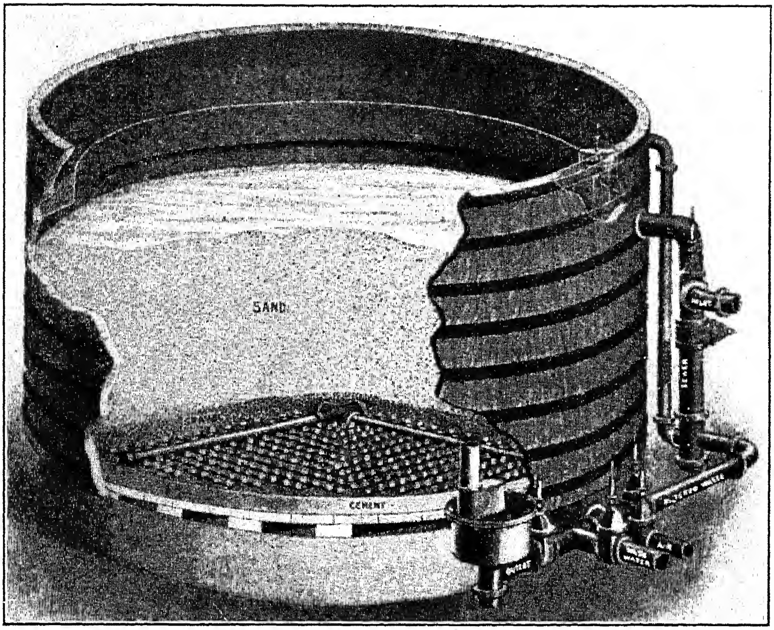


Fig. 41.—Early type of gravity rapid sand filter.

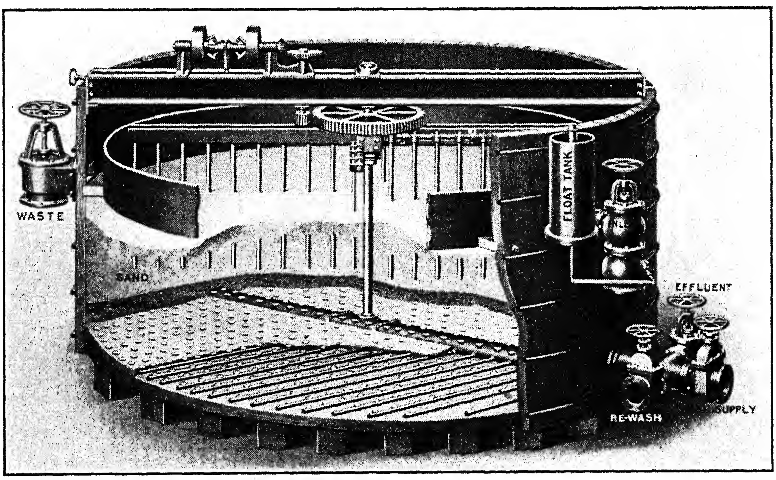
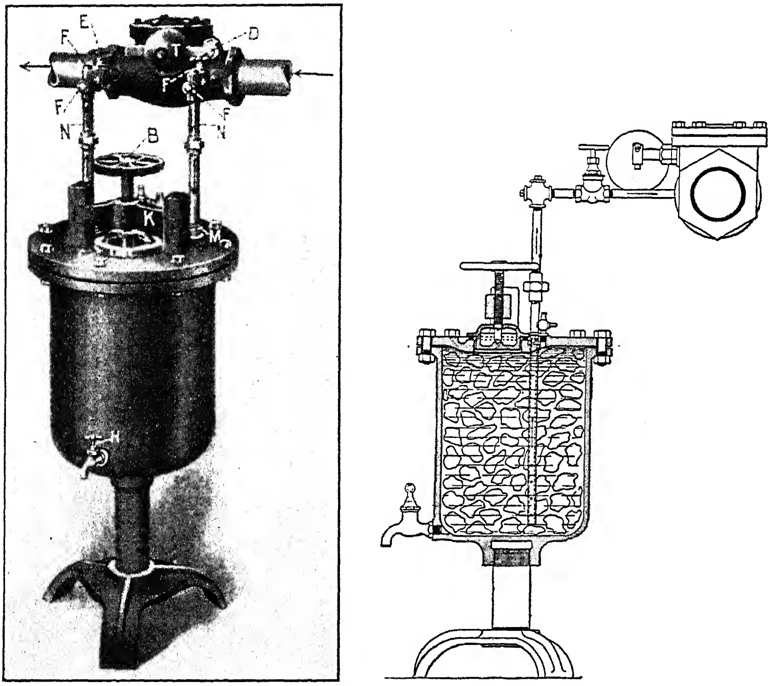


Fig. 42.—Gravity type of rapid sand filter, showing mechanical rakes.

One of the first methods employed in applying the chemical coagulant to the water which was to be passed through these filters, consisted of a box filled with alum crystals (Fig. 43) or with pieces of sulphate of alumina through which was bypassed a small quantity of the water on its way to the coagulation compartment or tank. The outflow from this chemical feed box was hand-regulated by means of a valve, and sometimes metered in



Roberts' Filter Mfg. Co.

FIG. 43.—Coagulant feed apparatus.

order to determine the rate of flow of the solution. There was no method of ascertaining the strength of this solution, and obviously its concentration was inversely proportional to the rate of flow through the feed box. In consequence the water to be treated was likely to receive its largest dosage of chemicals during the period when the volume of water flowing through the coagulation tank was the smallest and needed the least, and *vice versa*. Such crude methods of applying the chemicals have been improved upon in recent years for this type of filter, but are still frequently far from being satisfactory.

As pressure filters are closed receptacles (Figs. 44 and 45) and are usually operated under a pressure of 40 to 60 lb. per square inch, they may be located between the pumps, taking the water from the source of supply, and the filtered-water storage reservoir. Pumping directly through the filters to the supply mains themselves has also been used, and for small installations is still commonly employed. Pressure filters are useful in connection with large buildings, where open tanks are unde-

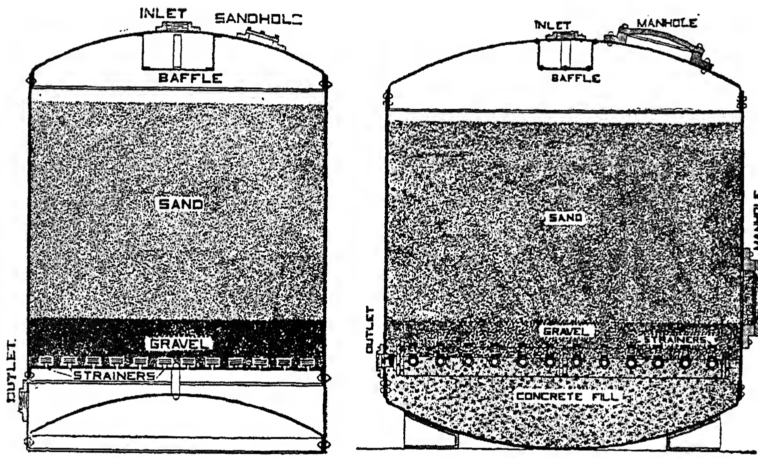
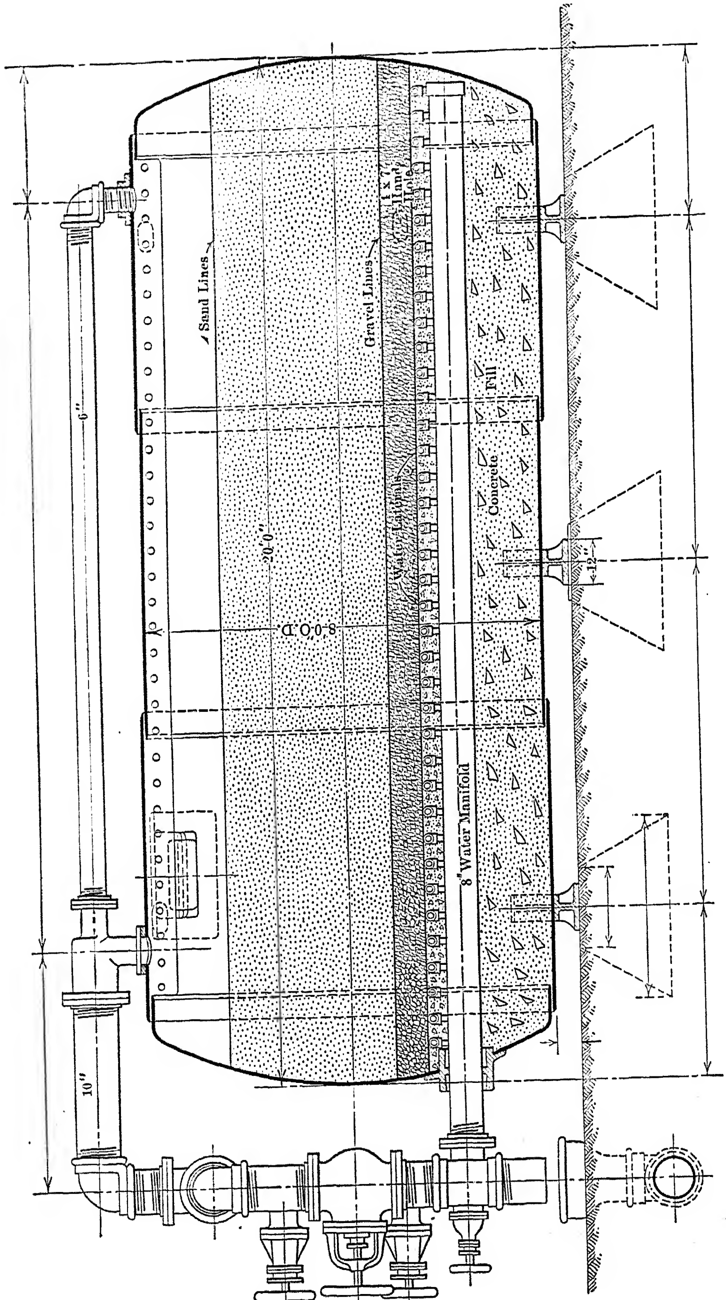


FIG. 44.—Vertical rapid sand-pressure filter.

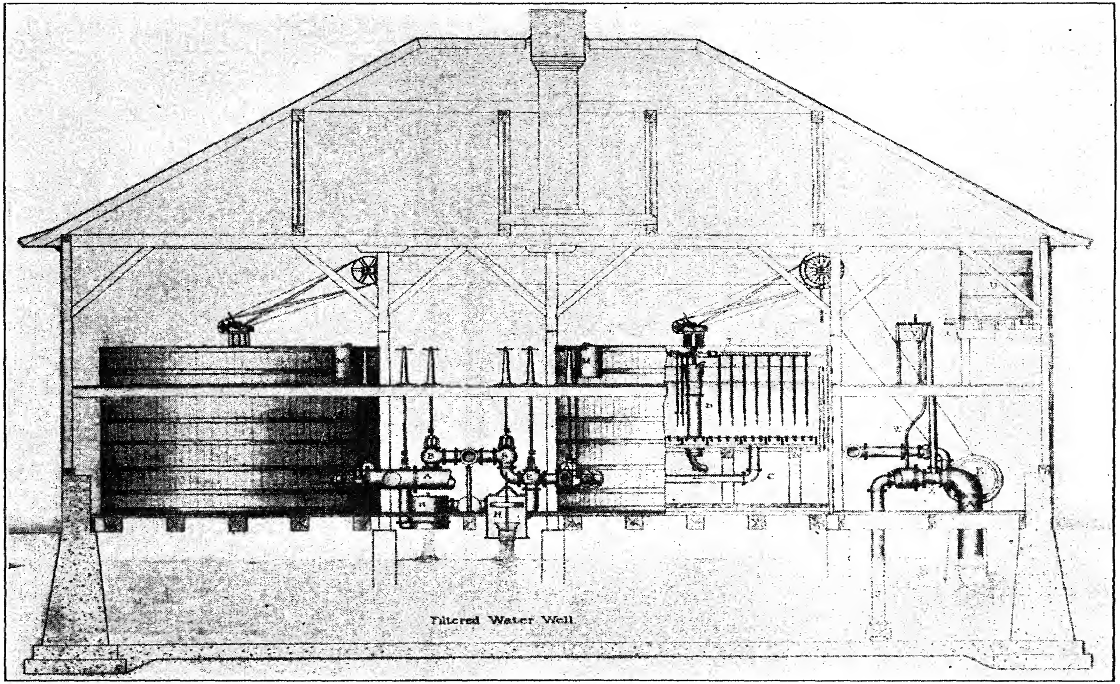
sirable. They are, however, neither efficient nor reliable from a sanitary standpoint, and are not economical to operate. They have not been generally adopted for purifying public water supplies, although quite a number of municipal plants were installed during the early development of this method of water purification. Their usefulness lies in the field of small plants for buildings, and where only a somewhat imperfect purification is permissible.

Gravity Filters.—As the name of this class of filters signifies (Fig. 46) the water flows through the sand by gravity. The first gravity filters to be constructed consisted of open cylindrical tanks of wood or iron. Some of them contained a compartment for permitting the chemical coagulant to act upon the water before being discharged on top of the filter bed, while other makers provided separate tanks for this purpose. Regulation of the rate of flow through the filter was usually accom-



Roberts' Filter Mfg. Co.

FIG. 45.—Horizontal rapid sand-pressure filter.



Reproduced by courtesy of New York Jewell Continental Filtration Co.

C, Sedimentation basin. D, Filter bed. A, Supply to filter. B, Wash to filter. E, Outlet for filtered water. H, Controller for regulating discharge of filtered water (butterfly valve not shown). J, Suction pipe for wash pump. K, Pump for washing filter bed. M, Float tank to regulate supply to filter. T, Agitating apparatus. U, Chemical tank. V, Alum feed pump, W, Alum feed pipe to filter. X, Propeller for operating alum feed pump. Y, Supply pipe for alum feed pump.

FIG. 46.—General arrangement of gravity rapid sand filters, filter piping and filtered water reservoir.

plished by means of hand operated valves. In some cases the water entering the filter was measured over a weir. Washing the filter bed was accomplished by forcing water upward through the bed, accompanied by a stirring of the sand by means of revolving rakes, the teeth of which projected downward into the bed. The movement of the rakes kept the sand bed loosened up, thereby permitting the wash water to come into contact with each particle of sand and preventing it from rising through the sand in separate channels.

These stirrers or rakes were driven by a small engine from a belted pulley and appropriate gearing, and were so arranged that

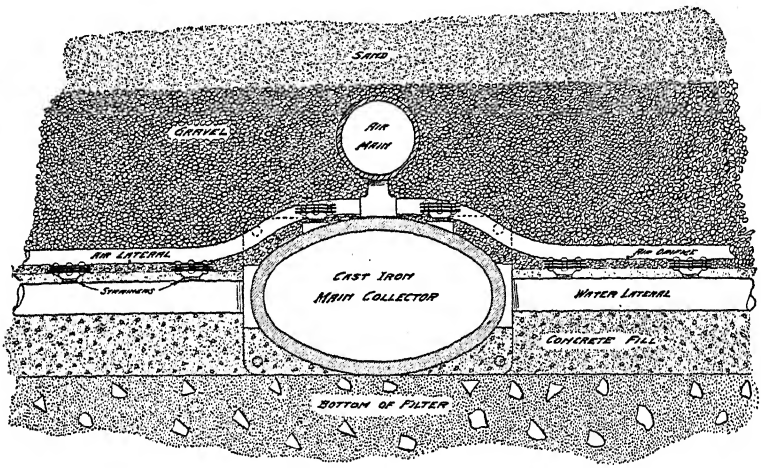


Fig. 47.—Arrangement of pipe manifold.

they could be gradually lowered as they revolved, and thus stir the sand bed to within a few inches of the bottom. The rake arms revolved at a rate of six to nine revolutions per minute, and the wash water passed upward through the bed at a rate varying from 7 to 9 gal. per square foot per minute.

The dirty wash water escaped through gutters or outlets from 7 to 12 in. above the sand surface. A common position for the gutter was around the inner circumference of the tank. By a suitable arrangement of the raw water-supply pipes, sewer pipes and valves, the waste-water gutters or outlets also served for the inlets for the treated water to be filtered.

The strainer systems and manifolds (Fig. 47) were varied in character, and presented, even as they do today, one of the most

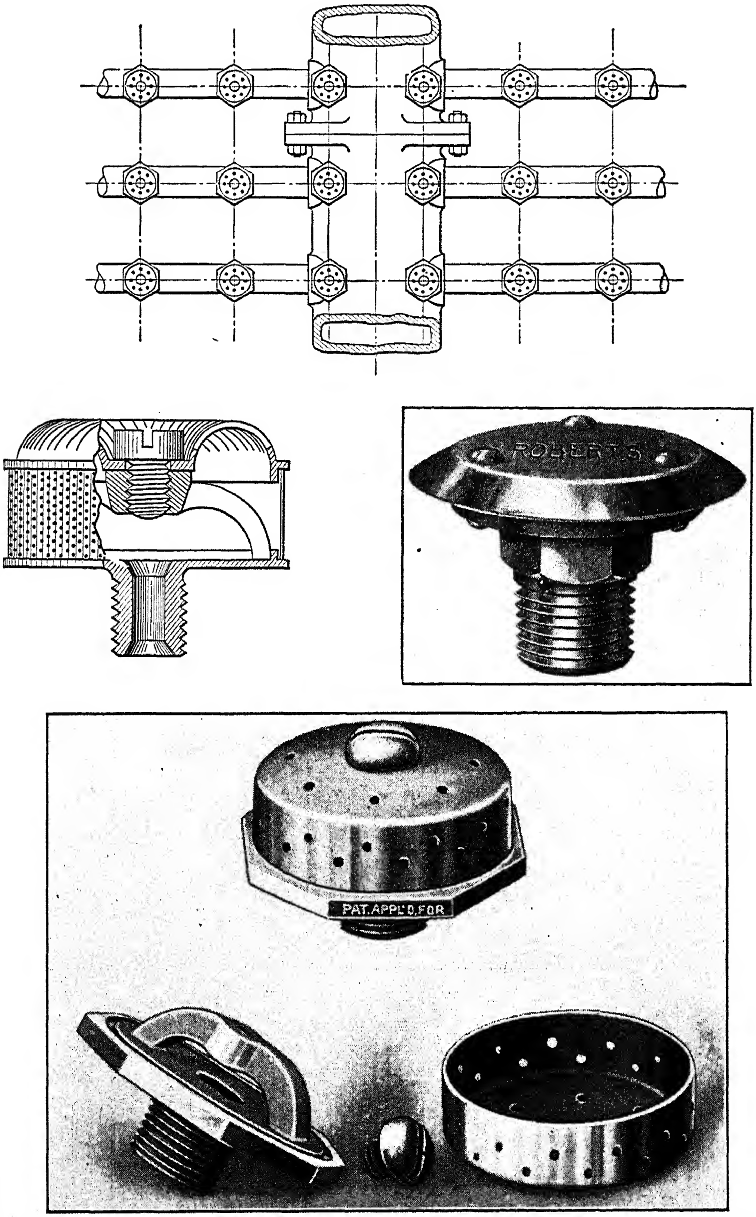


FIG. 48.—Types of strainers for rapid sand filters.

difficult and puzzling problems in the design of this class of filters. Slotted tubes, perforated plates, and strainer cups or nozzles were all used. The slotted tubes or nozzles were usually set in a manifold of pipes consisting of a central collector pipe with numerous laterals, usually extending at right angles to the main collector. Perforated plates or false bottoms were also employed. In some cases a false bottom fitted with nozzles was used. Many of these strainer systems are still in use, and the most modern of strainer systems are but modifications of the older types.

The method of applying the chemical coagulant to the water to be filtered in gravity filters was usually by means of small pumps. These pumps were sometimes automatically adjusted, where possible, to the stroke of the large pump supplying the water to the filter, but were more frequently hand-regulated. Solutions of definite strength were prepared in special tanks from which the chemical feed pumps drew their supply.

The early types of mechanical filters showed considerable ingenuity on the part of their designers in overcoming some of the difficulties inherent in a process which, though apparently simple, is in reality complicated. A knowledge of hydraulics and mechanics, to say nothing of the chemistry and bacteriology of the process, were necessary to solve most of the problems involved. In the rigid scientific examination and study of the process, undertaken between the years 1893 and 1901 by various investigators, were laid the foundations for the development of the modern rapid sand filter plant of today.

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CHAPTER XV

GENERAL ARRANGEMENT OF RAPID SAND FILTER PLANTS

In the development of rapid sand filters the necessity for properly preparing the water for filtration by adequate periods

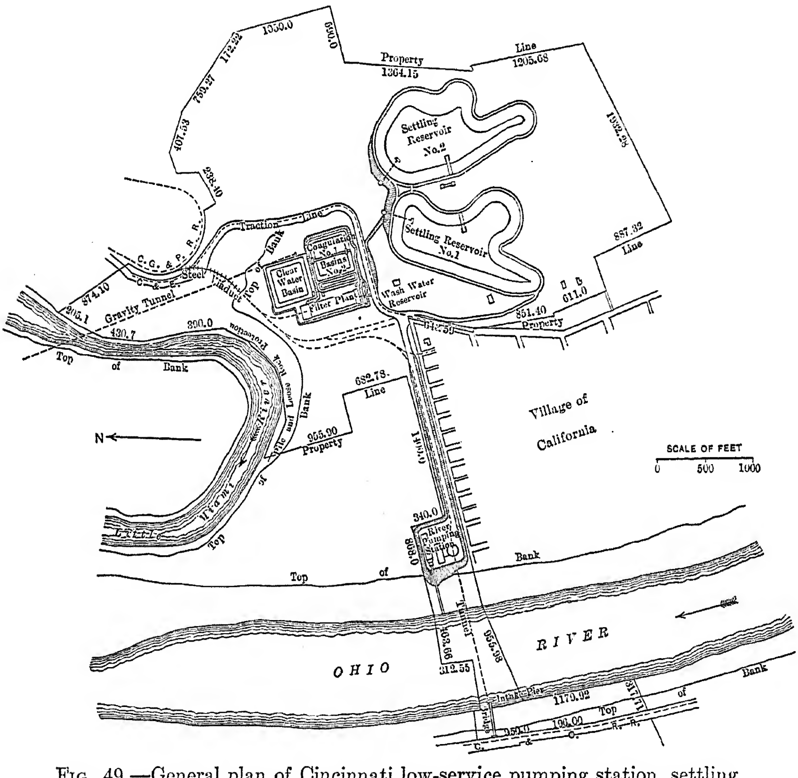

FIG. 49.—General plan of Cincinnati low-service pumping station, settling reservoirs and filter plant.

of settlement with coagulating chemicals soon made it evident that special basins of the proper size and shape for this purpose must form an integral part of the plant. Reservoirs or tanks for holding the filtered water were also found necessary, as a supply of the latter for washing the filters must be available. A modern filter plant (Fig. 49), therefore, is composed of at

least three distinct parts; *i.e.*, coagulation basins for the treatment of the water with chemicals; a set of filter tanks with the necessary system of piping, valves and controlling apparatus; and a filtered-water reservoir. A building is usually constructed over the whole or a part of the filter tanks. This building also generally contains the solution tanks for the chemicals, the regulating, measuring, and controlling apparatus for the application of the chemical solutions and the operation of the filters, the wash water pumps, and the laboratory and offices. The particular method of arranging and grouping the various parts of the plant is dependent to a great extent on local conditions.

Certain modifications of the above-mentioned parts of a rapid sand filter plant of the gravity type are necessary in the case of plants using pressure filters, but as the latter are not so generally employed for purifying municipal supplies, they need not be given further consideration at this point.

General Arrangement of Typical Plants.—Those plants which have to purify very turbid waters are usually provided with preliminary settling reservoirs in addition to the other parts of the plant mentioned above. No chemicals are used in these reservoirs and the precipitation of the sediment is effected by simply reducing the velocity of flow of the water as low as possible, thereby giving time for the suspended matter to drop out. These plain sedimentation reservoirs form an integral part of such plants as those, for example, at Louisville, Ky., and at Cincinnati, Ohio.

Several typical plain sedimentation reservoirs and coagulation basins have been described in Chapters VI and VII, and cuts are given showing their location in relation to each other and to the filters. It will be noted that the topography and other conditions of a purely local character usually determine the relative positions of the various parts of the plant.

Relative Position of the Coagulation Basins and Chemical House.—There are one or two special features in laying out the plan for a rapid sand filter plant to which it is desirable to pay considerable attention, and which, if incorporated in the plant, will make its operation less difficult. The inlet to the coagulation basins or to the mixing chamber, when the latter forms a part of these basins, is the usual point for the application of the coagulating chemicals. The preparation of the chemical solutions is generally carried on in a part of the filter plant building.

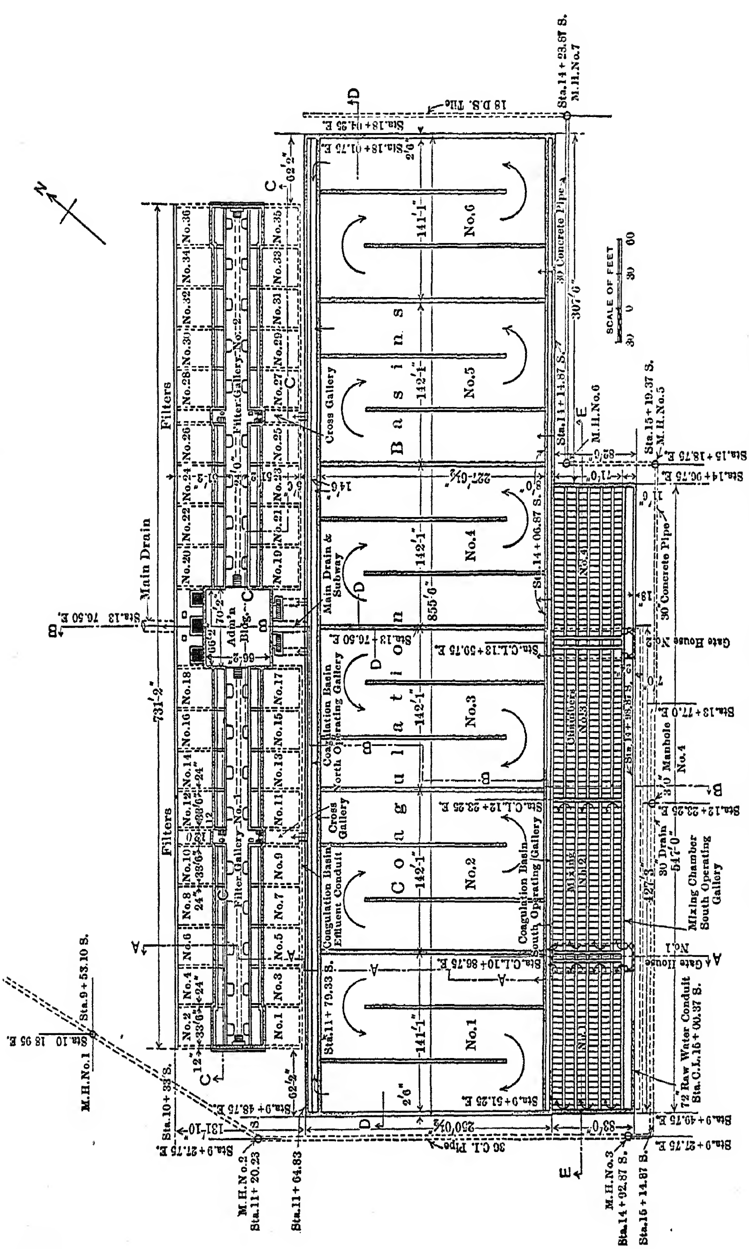

FIG. 49a.—General plan of Cleveland filtration plant.

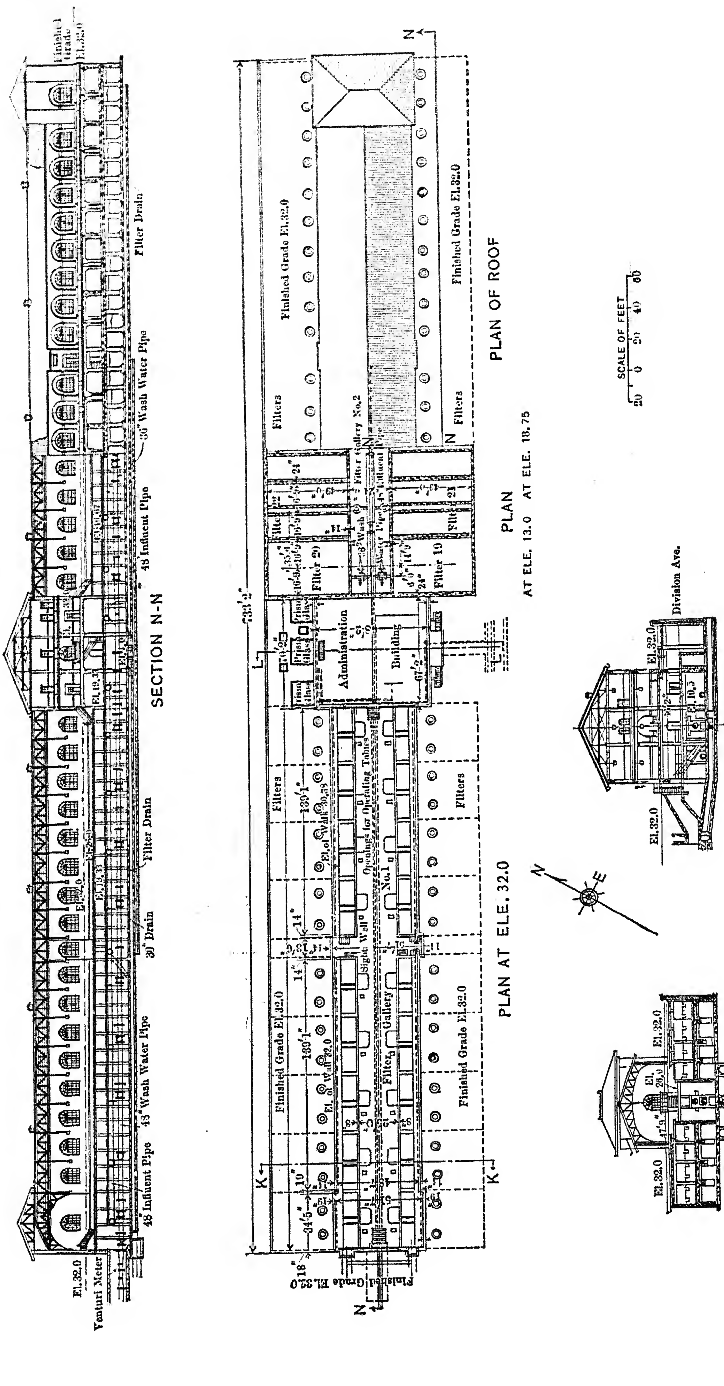

FIG. 49b.—Sections of Cleveland filtration plant.

The distance which these solutions must be conveyed should be as short as possible, in order to avoid the use of long pipe lines which may become choked up by deposits or by corrosion. These chemical feed lines should be readily accessible for their full length both for the purpose of cleaning or renewal.

In a plant where a large amount of chemicals are used, such as is the case where the softening of hard waters are made a part of the process, it may be best to place the chemical house over or close to the inlet to the coagulation basins, and quite apart from the filters and the filter house. This is the plan followed in the purification plant at New Orleans. At the Columbus, Ohio, water-softening and purification plant the design which was worked out resulted in a very compact arrangement of the chemical house, mixing tanks and coagulation basins. In the small plants a compact arrangement is more easily effected than in the large plants. For example, in the large plant at Cincinnati, Ohio, the chemical house had to be placed at a point convenient for receiving shipments of chemicals by rail. This happened to be a point located about 700 ft. distant from the place where the water to be treated could most conveniently be brought in order for it to be controlled in its passage into the coagulation basins.

Arrangements of Filters.—The filter tanks in the older plants were usually cylindrical in form, and were placed in two or more rows. This arrangement enabled a common water supply pipe to be laid between the rows of filters, as well as a common effluent pipe for the discharge of the filtered water. It also permitted the individual filters to be connected to a common drain or waste pipe to be used in draining the filters and when washing them. As the earlier plants were not provided with rate-of-flow controlling apparatus, the system of influent and effluent piping was comparatively simple.

Round wooden or iron filter tanks are not economical of space, and are also limited in the diameters to which it is practical to build them. With the development of concrete construction, oblong or square tanks of this material have taken the place of the round wooden and iron tanks. The former can be built side by side with no waste space between them, and are much more durable.

Concrete filter tanks are usually built side by side in rows, with a so-called pipe gallery between the rows In this space

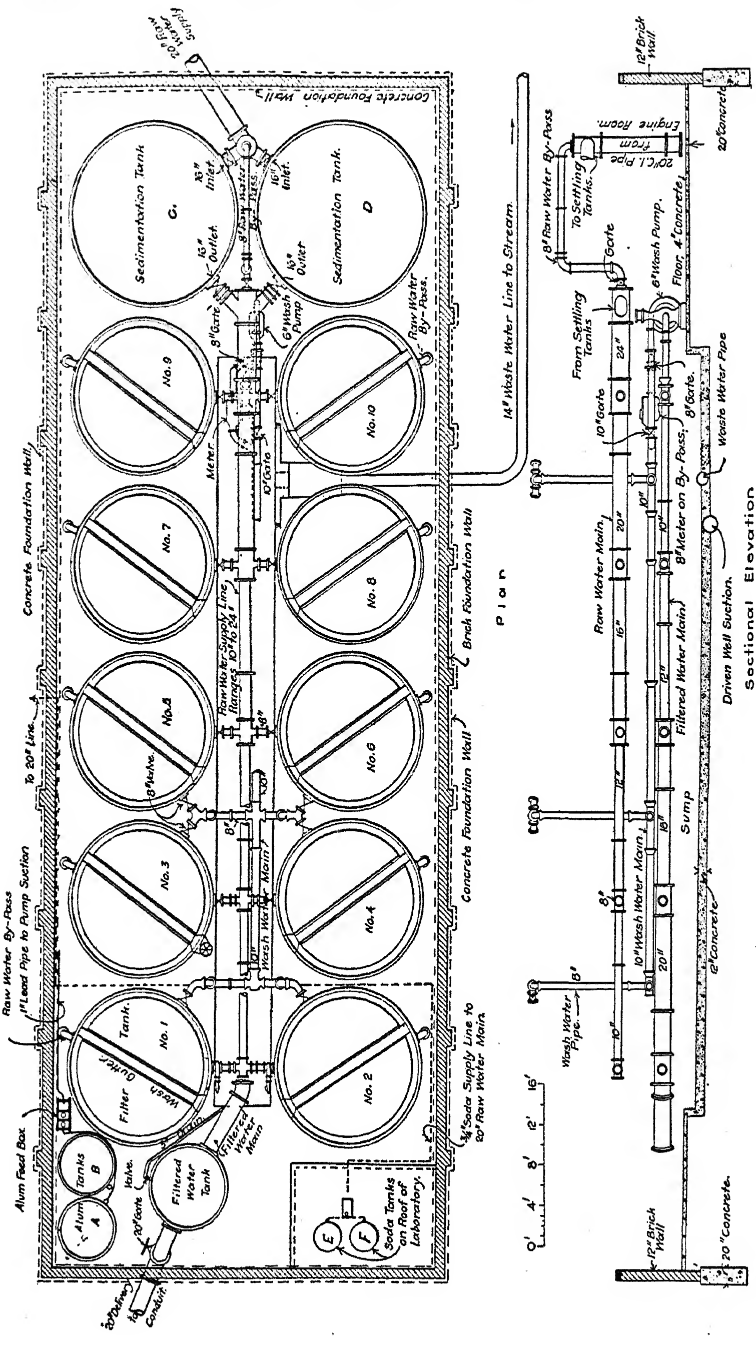

FIG. 50.—Mechanical filter plant at Baisley's Pond, Brooklyn, N. Y.

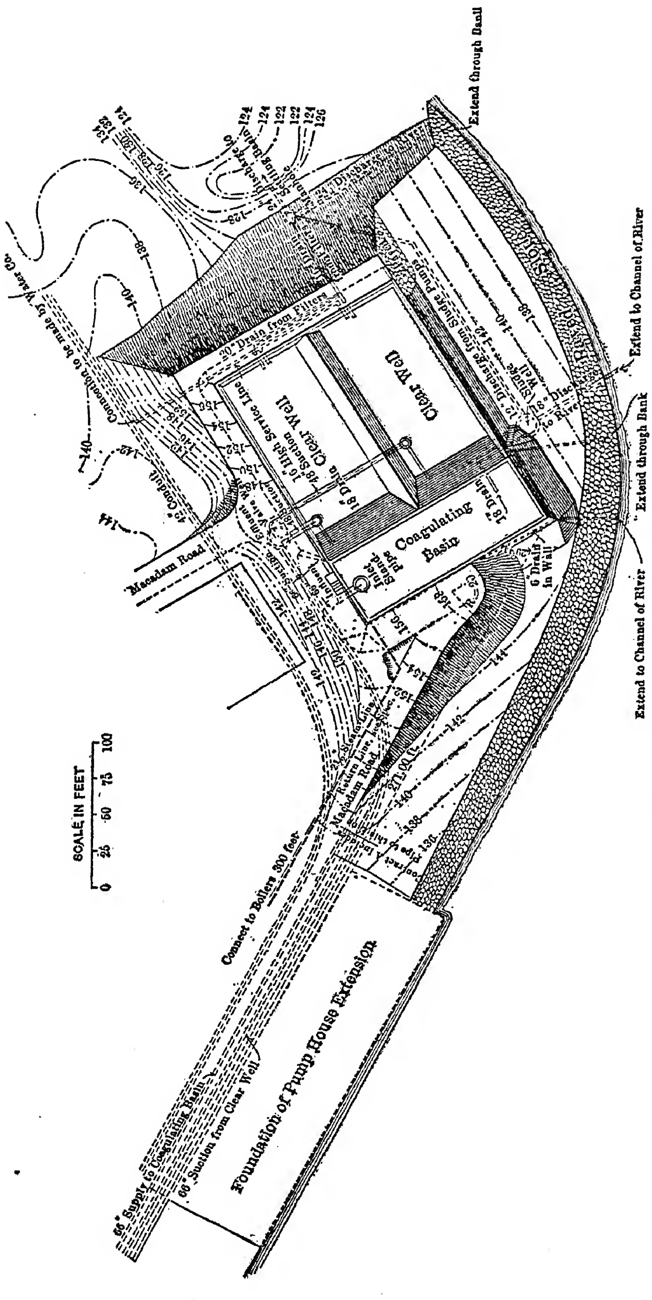

FIG. 51.—Little Falls, N. J., rapid sand filter plant.

are laid the influent and effluent piping or ducts. It furnishes space as well for the necessary valves, rate-of-flow controlling apparatus, gages, and so forth, required in operating the filters. An accompanying cut shows the old method of placing the round filter tanks (Fig. 50) and the requisite piping, while another cut illustrates the typical arrangement of a set of modern concrete filters (Figs. 51, 52 and 53).

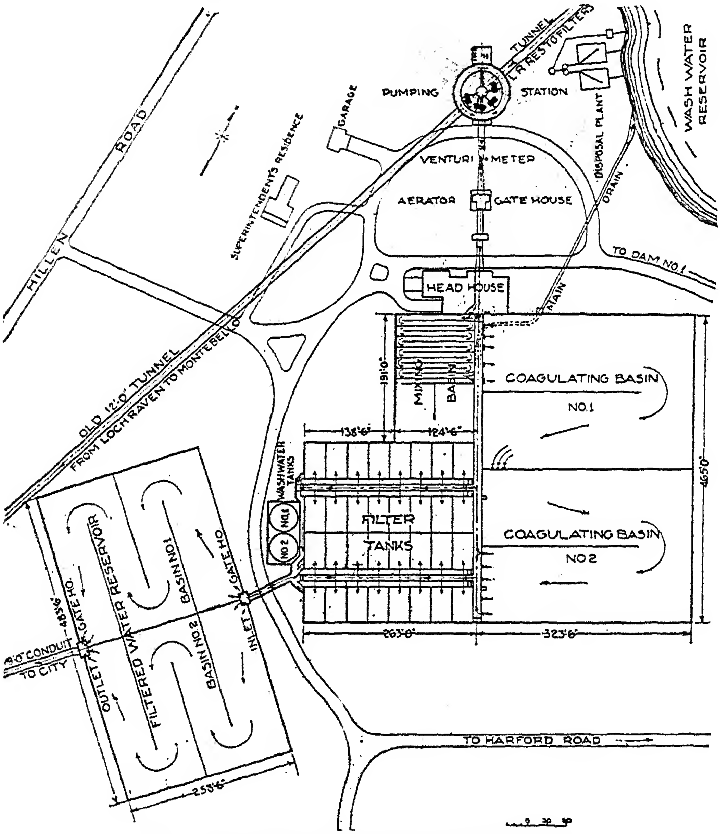

FIG. 52.—Baltimore rapid sand filter plant.

Another desirable feature in the arrangement of this type of plant is that the controlling, measuring and indicating apparatus should be located conveniently close together in the filter plant building. The apparatus referred to consist of the gages indicating the water levels in the various tanks, basins and reservoirs, the steam and compressed-air pressure gages, electrical

meters, water-meter registering devices, and valves for controlling the flow of water to the coagulation basins, filters and clear-water reservoir.

By thus grouping this apparatus fewer attendants are necessary, besides enabling the operator to have a better control over the purification process. Rapid changes in the method of operating plants of this kind are not infrequently required, and apparatus for controlling and indicating the effect of these changes should be centrally located. The larger the plant the more difficult, of course, does this compact arrangement become, but this feature should never be lost sight of in designing the plant.

Clear-water Reservoir.—The position of the clear-water reservoir in relation to the other parts of the plant is not so important as is that of the coagulation basins and filters. It is frequently placed under the filters, but may also be placed some distance away from them. The reservoir is usually covered in order to prevent growths of microscopic plants in the filtered water. In some of the large plants, however, the reservoir is not covered, but in such abundant growth of algæ and diatoms occur. The coagulation basins are sometimes covered in the smaller plants but are not usually so in the larger ones.

If the clear-water reservoir is not conveniently located for obtaining a supply of filtered water for use in washing the filters, provision is usually made for either drawing filtered water directly from the main effluent pipe from the filters, or from some pump well supplied from this main. In many modern plants separate wash-water storage tanks or reservoirs are provided for holding a supply of filtered water at a sufficient elevation to supply wash water under pressure during the washing of the filters, instead of by direct pumping during the washing process as is quite commonly the case in the smaller plants. This tank is usually located at an elevation of 50 or 60 ft. above the filters and is covered.

The filter-plant building generally contains the laboratories and offices of the plants, beside the rooms for the preparation of the chemical solutions, and the storage space for the chemicals. A heating plant must be provided for the filter-house building where the climate makes it necessary, and a power plant as well, unless steam or electric power is obtainable from some other source.

From the foregoing description of the principal parts of a rapid sand filter plant it can be seen that considerable latitude may be

allowed in their arrangement, but that certain important factors must be kept in mind in order that the various parts of the plant shall be properly coördinated. Local conditions will necessarily introduce factors which will require a modification of the ideal design, and which may even make an efficient and economical plant an impossibility. It, therefore, is the province of the designer to harmonize as far as possible the arrangement of the various parts of the plant in order that as good a plan as can be obtained may result under the conditions imposed.

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CHAPTER XVI

DETAILS OF RAPID SAND FILTER-PLANT CONSTRUCTION

The important parts of a rapid sand filter plant have been outlined in a general way in the preceding chapter. The present chapter will deal with the various portions of the plant in detail.

Number and Size of Filter Tanks.—The number of filter units in any given plant should be sufficient to meet the maximum demand for water in conjunction with such storage capacity as can be provided, and to permit of easy operation of the plant as a whole. At the prevailing rates of filtration for filters of this type, viz., 1.6 to 2.0 gal. per square foot of filtering surface per minute, there would be needed 433 and 348 sq. ft. respectively, of sand surface for each million gallons of daily output. A commonly used area is 350 sq. ft. of filtering surface for each million gallons of daily capacity. The larger filter units are usually constructed for either a daily capacity of 500,000, 1,000,000, 2,000,000, 3,000,000 or 4,000,000 gal. For municipal plants the gravity type of filter is not usually built of units under 500,000 gal. daily capacity, except for quite small plants, nor over 4,000,000 gal. even for the largest plants. Pressure filters are naturally limited in size, and are not usually built to exceed units of much over 500,000 gal. daily capacity.

The circular gravity filter tanks are constructed of either wood or steel, and of course vary in capacity according to their size. For a capacity of 150,000 gal. in 24 hr., the diameter of the tank is approximately 8 ft., while for a capacity of 500,000 gal. per day the diameter is about 15 ft. These tanks are from 7 to 9 ft. in height, although those which are combined with a settling chamber underneath may be as high as 16 ft

Pressure filter tanks are built of either cast iron or steel. Cast iron is used for the smaller sizes up to those which are about 2 ft. in diameter, while steel is used for the larger diameters. Pressure filters are built to be erected either vertically or horizontally. The vertical pressure filters range from 1 to 10 ft. in diameter, and from approximately 5 to 12 ft. in height. The horizontal pressure filters are usually constructed for the larger-sized units, and range from 10 to 25 ft. in length with a diameter of about 8 ft.

Concrete Filter Tanks.—Concrete filter tanks are constructed either square or oblong, and with open tops. They are frequently built over a filtered-water reservoir, in which case the tanks usually rest upon groined arches supported by piers whose footings are in the floor of the reservoir. Even where the space under the filters is not used for the storage of water, but merely contains the pipe collecting system of the filter underdrains, groined-arch construction is sometimes employed.

It is not uncommon to divide large concrete filter tanks into two parts by a central conduit. The latter serves to distribute inflowing water to the sand bed through a number of lateral gutters, which usually extend at right angles to the central conduit, and also to act as a collector for dirty wash water during the process of cleaning the filter sand. In large tanks tie-beams extending across the top of the tank give it rigidity, and may also act as supports to walls and covers above in those cases where only a part of the filter tank is within the filter building.

Waste-water Conduits and Lateral Gutters.—The central and lateral conduits within concrete filter tanks may be built of concrete or of sheet steel. The former material is generally used in the large tanks, and the latter in the small units. The central conduit may or may not be covered, but the lateral waste-water ducts are always uncovered. The lateral gutters are generally spaced from 5 to 7 ft. apart, in order that the wash water being flushed off the surface of the sand shall not have to travel at the most a greater distance than 2.5 to 3.5 ft. in order to escape to the central conduit. The edges of the lateral gutters should be of uniform height and smooth, since they act as weirs to the escaping wash water, and should draw from all parts of the bed at approximately the same rate. A departure from this type of gutter is found in the saw-tooth edged gutter, which consists of a series of V-shaped notches or weirs on either side of the trough, and which are intended to accelerate the horizontal flow of the dirty water to the gutter by providing a uniform series of restricted exits through which the dirty wash water escapes at a greater velocity than it could over the long weir furnished by the edge of the gutter of the usual type.

Lateral waste-water gutters are made with a number of different cross-sections, each of which is supposed to have some particular merit because of the shape selected. A common cross-section is a V-shaped bottom with vertical sides. The end

farthest from the central conduit is usually more shallow than the end next to it, as less water is carried by this portion of the gutter. Semicircular gutters are frequently used, especially when constructed of sheet steel. A cross-section of a concrete gutter, which has a shallow V-shaped bottom, and sides that incline from the vertical toward the center of the trough, is supposed to prevent loss of sand during the washing process. Another type provides wings, which project out from the upper edge of the gutter, and are designed to prevent sand being carried over the edge, especially where air is used in agitating the sand bed prior to or during the application of wash water. Air vents are placed at intervals along the edge of the gutter, which allow the escape of air used to agitate the sand and which would otherwise become pocketed underneath the wings.

The carrying capacity of lateral gutters is frequently too small for the volume of wash water which must be conveyed away. Flooding of these gutters represents a loss of efficiency in washing the filter and should be avoided by providing them with ample carrying capacity. The spilling of the water along both sides of the gutter and for its full length, as in the usual type, produces so much loss of head in the water flowing in the trough that the usual computations for flow will indicate a size and pitch for the gutter which are much too small.

In an appendix (B) will be found a formula for calculating the approximate discharge of water from wash troughs of rapid sand filters. This formula has been derived by Mr. C. N. Miller, Assoc. M. Am. Soc. C. E., from a consideration of the laws of hydraulics, and from certain experimental data obtained by the author, which permitted a constant for the formula to be determined.

Influent Piping and Conduits.—It is undoubtedly the best practice if possible to so arrange the level of the filter tanks with reference to the coagulation basins, that the coagulated water can flow directly to the filter tanks with the least possible difference in head. Check valves, of any kind which are intended to regulate the discharge of the water applied to the filters, are frequently so irregular in their control of the flow that they permit a sudden rush of water onto the bed and thereby disturb the filtering surface. A large central wash-water conduit in the filter tank may act in a measure as a stilling chamber, but it is better not to rely on such an effect if it is possible to provide flow lines that will

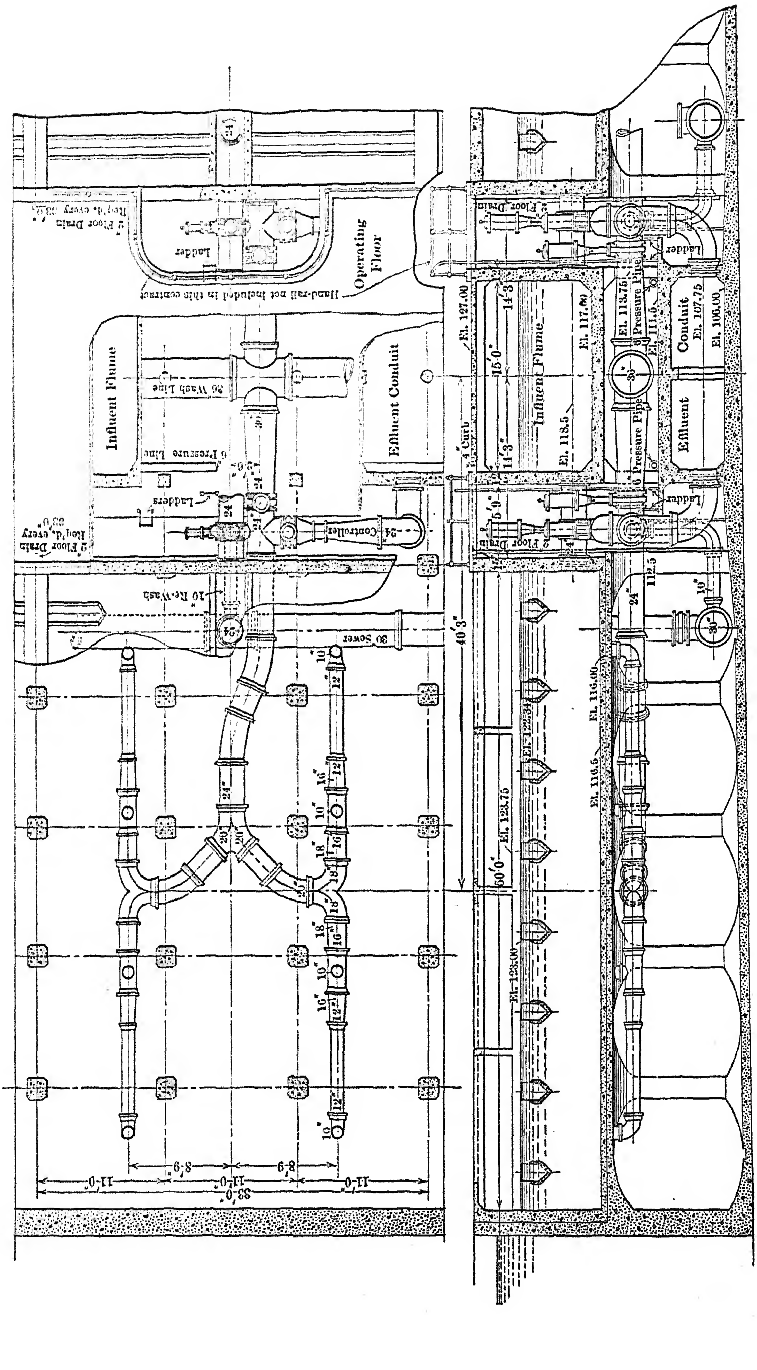

FIG. 55.—Plan and cross-section of filter piping and pipe gallery in St. Louis filter plant.

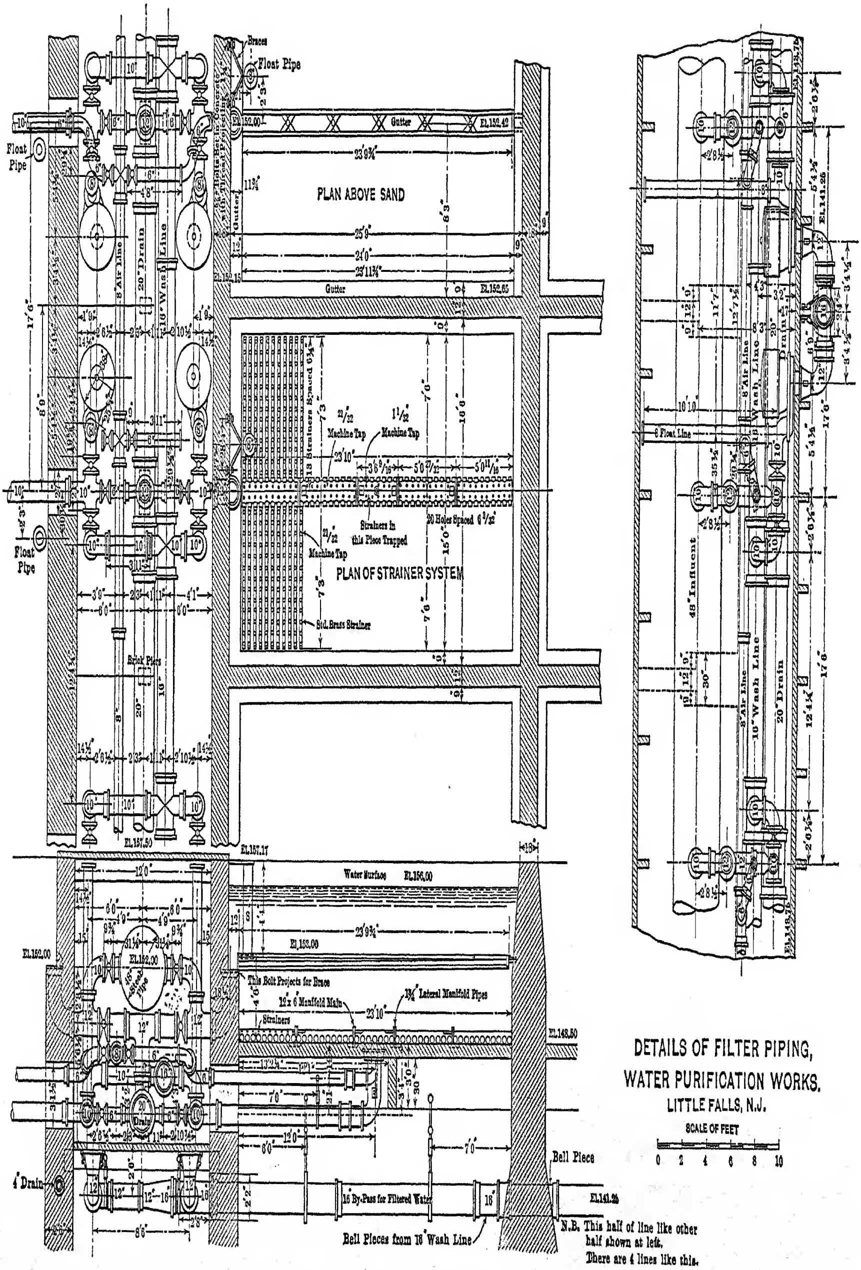

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FIG. 54—Cross-section of filter piping and pipe gallery in Little Falls, N. J., rapid sand filter plant.

permit the influent valve to the filter to be left wide open while the filter is in service (Figs. 54 and 55).

Since the pressure upon the influent and effluent piping of filters of the gravity type is low, light cast-iron pipe may be used. In some modern plants concrete conduits have displaced the cast-iron influent and effluent pipe lines entirely, except for short connections where valves must be inserted. Where these concrete conduits can be made water-tight and where satisfactory connections between them and the pipe that must be used can be made, there is probably some economy in this method of construction. However, the cracking of such conduits due to expansion and contraction is not uncommon, and appears to be most likely to occur where wall nipples connect with valves or with short pipe. Where there are double- or triple-deck conduits, the upper one carrying coagulated water, the next lower conveying filtered water, and the lowest duct carrying away dirty wash water, as is the case in some plants, especial care is necessary to make the conduits water-tight, since contamination of the filtered water by leakage from the upper or the lower conduit into the middle conduit is not impossible under certain conditions.

Effluent Piping and Conduits.—The system of pipes or channels which serve to collect the filtered water from the underdrains of the filter tank differ more or less as to their design. Cast-iron piping is commonly used with the large concrete filter tanks, while wrought-iron piping is more easily adapted to the small circular wooden and steel tanks. In large concrete filters the underdrain system is sometimes divided into sections, each part being served by a large single or branched collector pipe, which is connected to larger pipes, and they in turn with the effluent main in the pipe gallery. Another method is to construct large concrete collecting channels as a part of the underdrain system. The main channel collector is connected with the effluent pipe in the pipe gallery directly, and thus does away with a system of collecting pipes underneath the filter tank.

Since the effluent pipes (Figs. 56 and 57) or conduits must serve, in conjunction with the underdrain system, both as a collector of the filtered water during filtration, and as a distributor of wash water during the washing process, they should be so designed as to keep the head lost in the flow of water at a minimum. Relatively large pipes or conduits (Fig. 58) are required to produce this effect, and much care is necessary to proportion them prop-

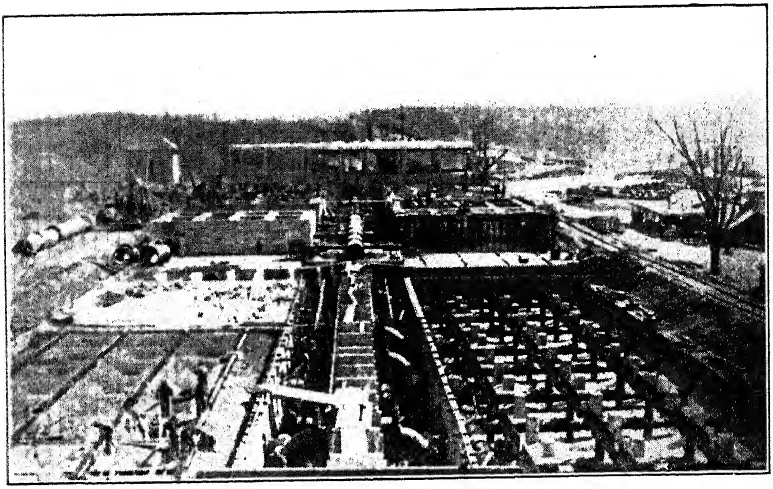

FIG. 56.—Pipe gallery and under drain system of Cincinnati filters.

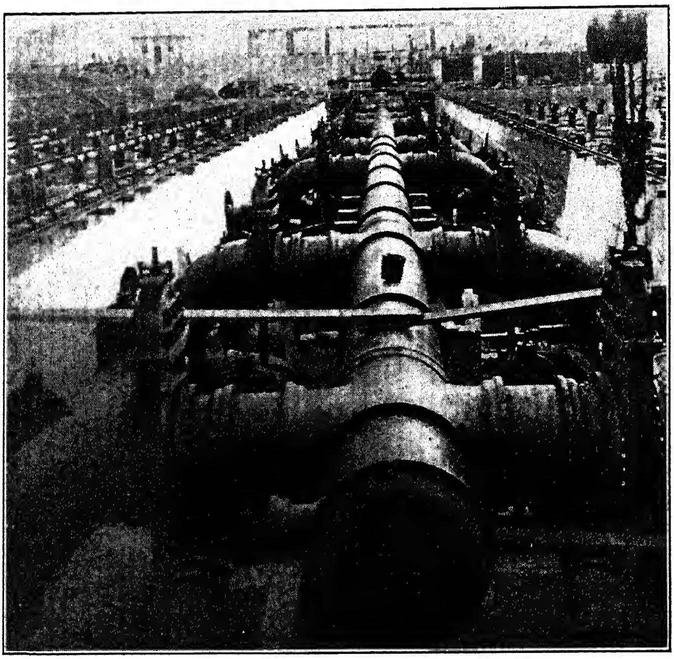

FIG. 57.—Wash water piping of Cincinnati filter plant

erly. Any lack of uniformity in the draft on all parts of the filter during filtration is undesirable, and any condition which prevents a uniform delivery of wash water to all parts of the bed must be avoided. Next to the strainer system, the collecting system of pipes and conduits forms the most important and the most difficult part in the design of rapid sand filters.

In the main discharge line from the filter is now placed in all the large and well-designed plants a regulator in order to control the rate of filtration. This may be a separate device through which the water flows, or it may be an auxiliary mechanism which utilizes the effluent valve to regulate the flow. When the

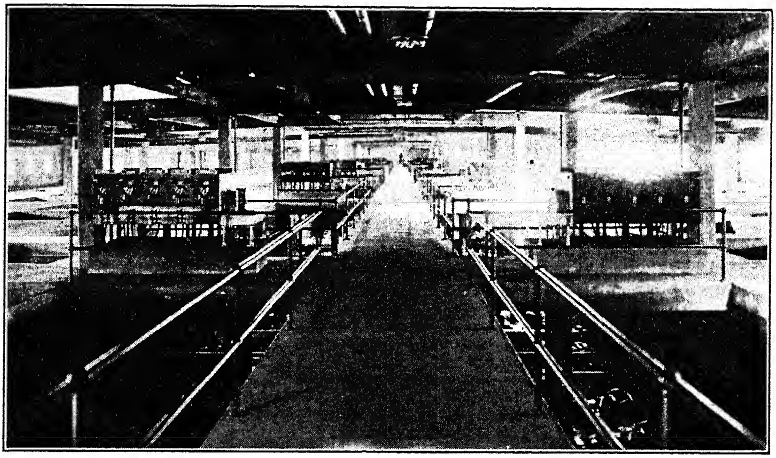

FIG. 57*a*.—Interior view of filter house in Cincinnati filtration plant.

rate controller is an integral part of the effluent pipe line, it is placed just back of the effluent valve which cuts off the filter from the main filtered water pipe line or conduit. The design and operation of these regulating valves will be considered under a separate section.

Waste-water Piping.—Since the discharge of dirty wash water in all filters is through some system of troughs or gutters, it is only necessary to connect such a system with a main waste pipe which will convey the dirty wash water to the sewer. Where a central wash-water conduit forms a part of the filter tank, the escape of the wash water is usually through the bottom or end of the conduit to a pipe line leading directly to the sewer. In this line is placed the waste-water valve. It is also customary to make a

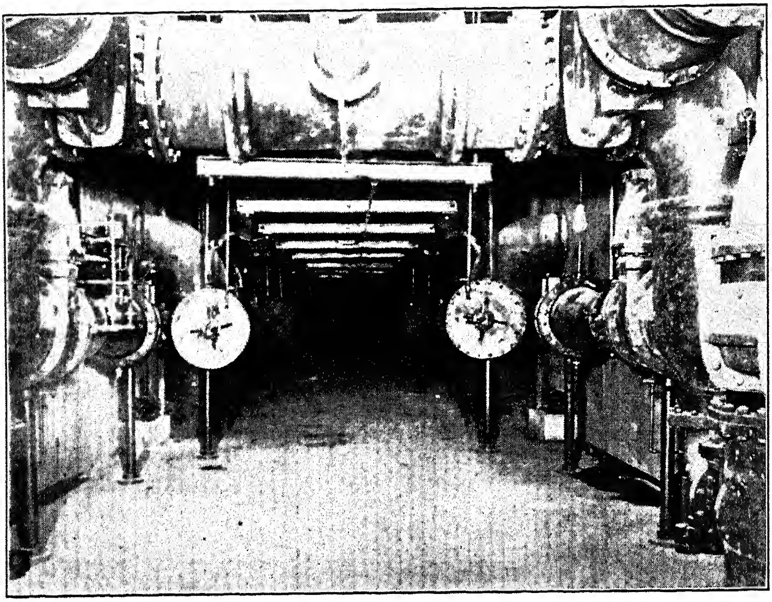

FIG. 58.—Pipe gallery of Minneapolis filter plant.

FIG. 58a.—Operating floor of Minneapolis filter plant.

cross-connection between this pipe line and the filtered water pipe of the filter tank, and to provide it with a separate valve. This connection is for the purpose of draining the filter directly to the sewer, when necessary, without allowing the drainage water to pass into the filtered-water pipe line.

Wash-water Piping.—Since the underdrain system for carrying away the filtered water also serves as the distribution system for the wash water, it is only necessary to provide an independent connection between the main effluent pipe from the filter and a separate pipe line for supplying the necessary wash water under pressure. The pipe connection is made back of the rate controller

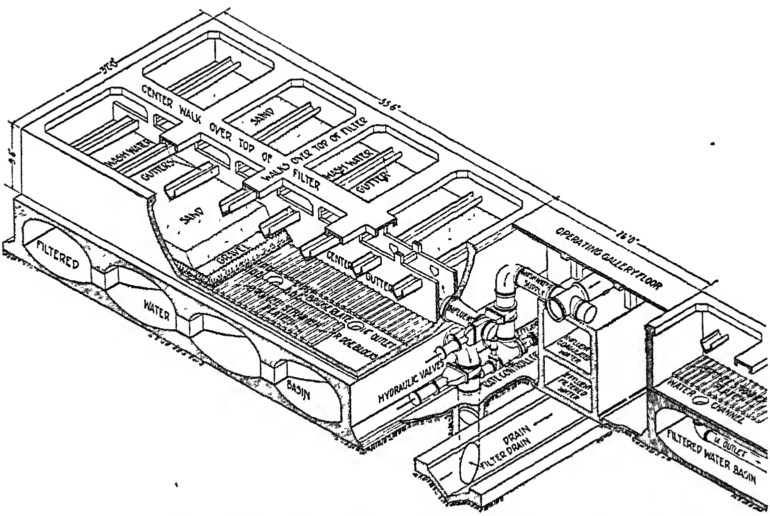

FIG. 59.—Isometric view of filter unit in Baltimore filter plant.

when the latter forms a part of the pipe line. The connecting pipe must, of course, be provided with a valve.

Air Piping System.—A method of agitating the sand bed during the washing process consists in forcing compressed air through the water underdrain system of the filter tank, and thence up through the strainer system, gravel and sand beds, or through a separate system of pipes usually laid on the top of the gravel layer and under the sand bed. This compressed-air piping is sometimes carried over the top and down into the filter tank from above. The main air supply pipe is brought through the pipe gallery with the other large pipes and conduits which supply or withdraw water from the filter tank.

Pipe Gallery and Cellar.—From the foregoing description of the piping and conduits with which it is necessary to equip each of the several filter tanks of a rapid sand filter plant, it is evident that considerable space must be allowed between the rows of tanks in order to provide a place for them (Fig. 59). If piping passes out through the bottom of the tank, it should be accessible, and in consequence a cellar is sometimes provided which contains nothing but this underdrain piping. The space underneath the tanks is also frequently utilized as a filtered-water reservoir. This is especially common in the construction of the smaller filter plants.

FIG. 60*a*.—Operating floor of Cleveland filter plant.

The pipe gallery (Fig. 60) should be large enough to accommodate all the main and connecting pipe lines, and allow enough space for making repairs and the inspection of piping and valves. Considerable ingenuity is oftentimes required in laying out a pipe gallery plan so that all the pipes, conduits and valves are included that are desired, and so that there are no interferences, and yet provides the extra space needed for inspection and repairs. Several typical plans and cross-sections of pipe galleries are shown in the accompanying cuts

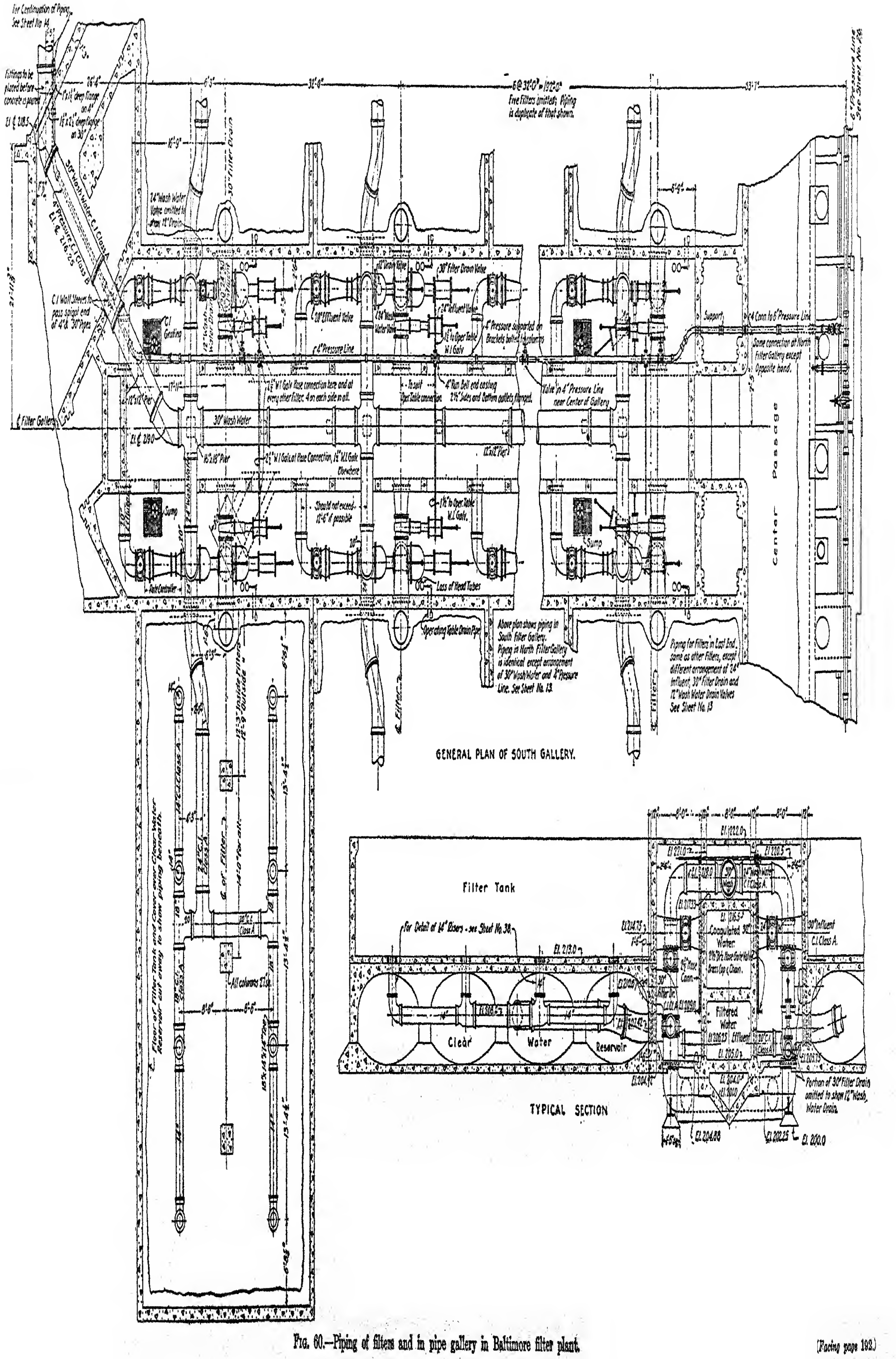

FIG. 60.—Piping of filters and in pipe gallery in Baltimore filter plant.

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Valves.—Each filter tank is usually provided with five valves, viz., an influent valve for admitting the water to be filtered, an effluent valve for stopping the flow of filtered water from the tank, a wash-water valve for permitting the application of wash water to the underdrain system, a drain valve for allowing the escape of the dirty wash water, and a waste or "rewash" valve for wasting the filtered water after its passage through the tank, when it is desirable to do so. The first four valves are essential to the proper operation of the filter. The fifth valve or waste-water valve is not absolutely necessary except where it is desired to drain the water from a tank and from the underdrain system without permitting it to pass out through the effluent valve into the

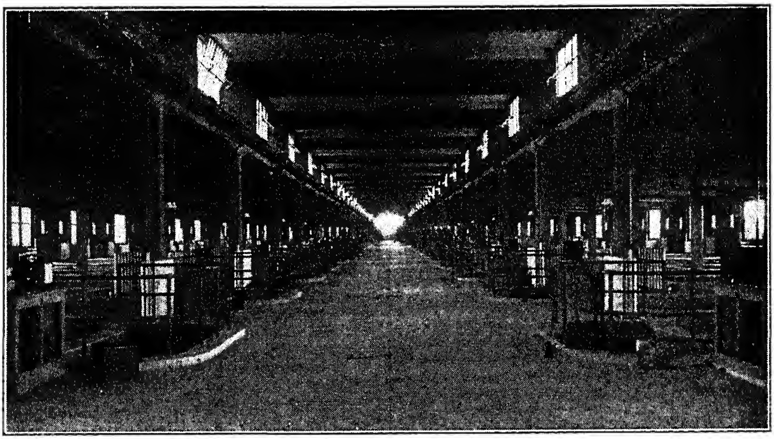

FIG. 60*b*.—Operating floor of St. Louis filter plant.

filtered-water main. It is commonly provided for in a system of piping and valves and may be found useful at times, although as a rule it is little used.

In small plants the filter-tank valves are usually hand-operated. They are as conveniently placed as possible on the various pipe lines and are turned by means of hand wheels. Where the valves must be located in a pipe gallery at some distance below the top of the filter tank, extension stems with hand wheels attached are employed.

In gravity filters it is important that the operation of the filter during both the processes of filtering and washing should be observed, and that the various valves controlling the movement of

the water into and from the filter tank, should be easily and quickly accessible to the operator. The opening and closing of valves must usually be done rapidly. This is not possible with large valves when hand-operated. Because of these conditions in operating and because observation of the processes of filtering and washing must take place at the top of the tank, power operated valves, whose movements are controlled from a table or switchboard, have come into general use especially in the larger plants. Hydraulically and electrically operated valves are both used, but the former are much more extensively employed than the latter.

Hydraulic Valves.—The hydraulically operated valve consists of the usual body of the valve proper, surmounted by a closed cylinder containing a movable piston (Fig. 61). This piston is

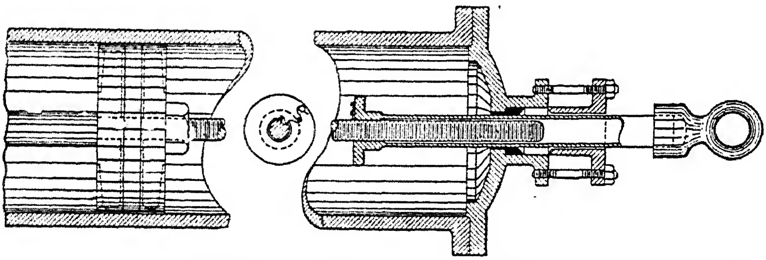

Fig. 61.—Piston of hydraulic valve showing adjustable stop for use on wash-water valves.

attached to a rod connected to the movable gates of the valve. Pipe outlets at the top and bottom of the cylinder permit water under pressure to be admitted to either side of the piston. The hydraulic pressure is usually controlled through a multiple-way cock, which when set in one position admits water to one side of the piston and releases it on the opposite side or *vice versa*, thus moving the gates of the valve to either an open or closed position. The small pressure pipes with their multiple-way cocks are usually brought to a table or board located at one end and at the top of the filter tank, where the handling of the valves and the observation of the processes in the filter can be conveniently conducted.

The position of the hydraulic cylinder of the valve should, if possible, be horizontal, or at least at some inclination from the vertical. In the vertical position any leakage which took place around the piston, and which reduced the pressure sufficiently to allow the piston and gates to drop by gravity, might close the

valve, although it had been left open. The position of the valve gates is usually indicated by a hand moving over a dial. The hand is operated by cords passing through small pulleys and connected to the movable piston. The pressure piping and multiple-way cocks should all be of brass, as corrosion should be avoided, if possible.

Mr. George W. Fuller states[1] that

"For filter service it is rather desirable to specify a double-gate parallel-seat valve, as wedged-seat valves for this type of service are more likely to stick. Valves should be bronze-fitted throughout and the operating cylinders should be bronze-lined. Operating pistons can to best advantage be fitted with double-cup leathers. In determining the size of the operating cylinder, good service may be obtained by making the cylinder of such size that under normal conditions the total effective moving force on the piston is equal to the total pressure on the valve disk, assuming this in no case to be less than 30 ft. This is roughly equivalent to the assumption that the coefficient of friction of the disk, including the friction of the stuffing boxes and piston packing, is approximately 100 per cent."

Hydraulically operated valves usually give little trouble if properly designed and set, and if kept in constant use. Sticking of the piston in the cylinder sometimes occurs when the valve is left unused for some length of time. Ordinary care and attention should be given the moving parts, stuffing boxes, etc., and any corrosion of bronze parts of the cylinder or rod carefully noted, and, if possible, remedied. Sticking and leaking of brass multiple-way cocks give more or less trouble, but well-designed cocks may be had which reduce such difficulties to a minimum.

Electrically Operated Valves.[2]—Either direct (Fig. 62) or alternating-current driven motors may be used in operating valves of this type. Direct-current motors for operating valves are built with compound-wound field coils, or as plain series motors. If compound-wound just enough shunt field winding is employed to prevent the armature from reaching a dangerous speed, if operated with no load. This type of motor gives a high starting torque compared with its normal rating, and is well suited for

[1] George W. Fuller: "Some Features of Detail in the Design of Rapid Sand Water Filtration Plants." *Engineering-Contracting*, Sept. 2, 1914.

[2] The author is indebted to Mr. J. S. Gettrust, Assoc. M. Am. Soc. E. E., for certain descriptive matter pertaining to electrically operated valves and registering apparatus.

valve operation as the motor slows down and exerts a high torque when most needed in starting, and speeds up as the load decreases.

Direct-current motors offer another advantage in the operation of valves in that they may be stopped very quickly by cutting off the motor from its supply of current and connecting a resistance across the terminals of the armature. The motor then becomes a generator, and the inertia of the moving parts is used in forcing current through the resistance where it is dissipated as heat. This is known as dynamic braking, and, when properly effected

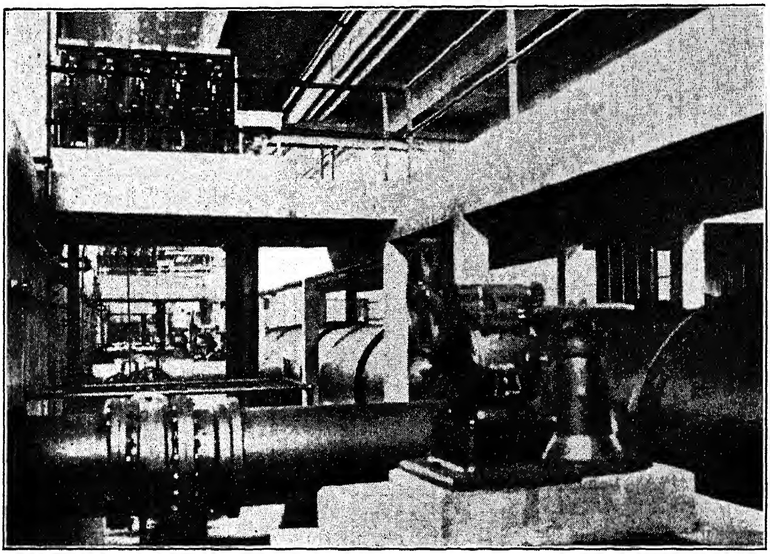

Fig. 62.—Electricially operated gate valves at Cincinnati filter plant.

by automatic devices, enables the gates of the valve to be stopped at any desired point. It is thus possible when closing a valve to seat the gates without danger that the inertia of the moving parts will cause the gates to overtravel and become wedged in the seat.

In the accompanying cut (Fig. 63) is shown a control panel for a direct-current motor used in operating large valves. The motor can be made to move the valve in either direction by throwing the four pole-double throw switch to either side, and then operating the movable contact arm. The two lamps are used to indicate whether the valve is closed or open. The panel is equipped with fuses, an overload circuit-breaker, no-voltage

release, and shunt trip coil, which latter is operated by a limit switch geared to the motor. In Fig. 64 is shown a type of control panel by which the valve gates may be stopped at any intermediate point between a wide-open and fully closed position by means of a dial contact switch placed on the panel just under the lamps.

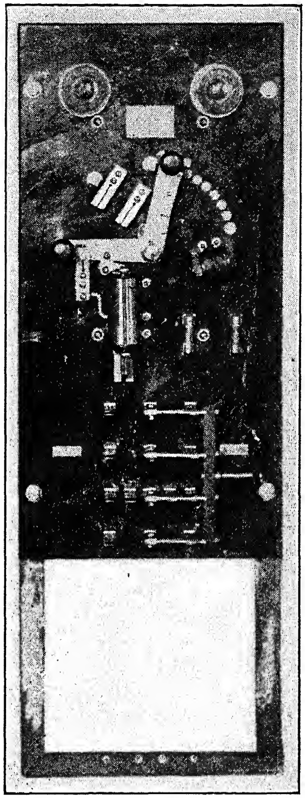

FIG. 63.—Panel from which direct-current valve motors may be operated.

For automatically stopping the travel of the valve gates various forms of limit switches have been used. They consist essentially of a set of movable contacts which are operated by a screw shaft, which is geared to the valve. The contacts close circuits at either end of their travel which operate the mechanism for disconnecting the motor from the power circuit. The movement of these circuit-closing contacts is proportional to the movement of the valve gates. Some electrically-driven valves are operated without limit switches. The increased load on the motor, as the valve reaches the limit of its travel, operates an overload coil, which opens the controller circuit. This type of apparatus requires a slow-speed motor and a special construction of the valve.

Experience has shown that the tripping mechanism for opening the motor circuit, when the valve reaches the limit of its travel, is not always reliable. If the circuit is not opened the valve stem may be twisted and bent, or some part of the equipment such as the gear teeth, motor frame or valve bonnet, broken. By installing additional contacts on the limit switch which will short-circuit the motor in case of the failure of the limit switch to act, this difficulty may be overcome. These contacts must be accurately set, and when so placed will promptly blow the fuses in the motor

power line if the limit switch does not act, and thereby prevent more serious damage to the valve and motor.

The polyphase squirrel-cage type of motor is the most simple form of motor for operating valves with an alternating current. It may be built to operate even when submerged in water. The characteristics, however, of this type of motor do not lend themselves as well to the operation of valves as do those of the compound-wound direct-current motor. As their starting torque is

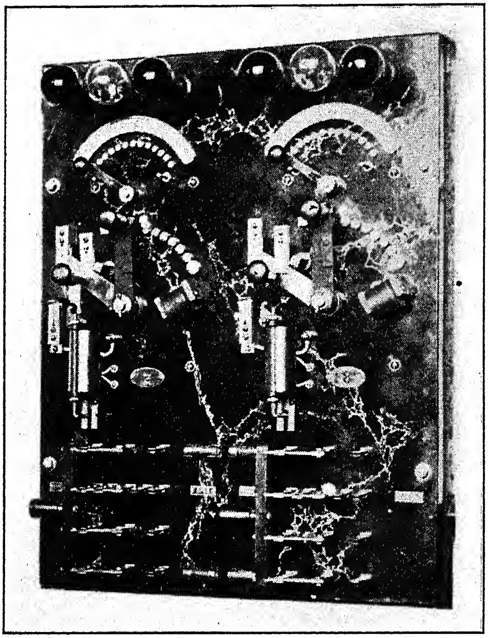

Fig. 64.—Panel from which direct-current valve motors may be operated, and so arranged that valve may be stopped at intermediate points between wide open and shut.

low as compared with a direct-current compound-wound motor, it is necessary to design them especially for a high starting torque, which necessarily involves the use of a larger motor than would be the case if a direct-current motor was used.

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CHAPTER XVII

DETAILS OF RAPID SAND FILTER-PLANT CONSTRUCTION (CONTINUED)

STRAINER SYSTEMS

The strainer system of the filter tank serves the double purpose of collecting the water which has passed through the sand and gravel beds during filtration, and of distributing the water applied to cleanse the filter bed during the washing process. As the velocity of flow of the water is so much greater in the latter process as compared with the former, it becomes the governing factor in the design. A uniformly distributed upward flow under the whole area of the filter bed is the essential feature of successful washing, and the efforts of designers to approximate such a condition are clearly shown by the history of the development of this part of rapid sand filters.

The simplest form of strainer system is a perforated false bottom of metal on which the filtering medium rests. Fine wire-cloth screens were first used for this purpose, but their failure was usually due to the clogging of the screen with sand, which rested directly on it, and to uneven distribution of the wash water by the screen. As the size of the filter units increased these difficulties became more pronounced. When the false bottom was constructed of brass wire cloth or of perforated sheet brass, the cost of the strainer system was considerable and became a factor of importance with the increasing size of the filter tanks.

To overcome uneven distribution and to reduce the cost, small hemispherical or otherwise-shaped brass nozzles or caps, which were either perforated with small holes or provided with very narrow slots, were used. These nozzles were screwed into a false bottom made of either wood or iron, or into a manifold of iron pipes (Fig. 65) which formed a grid on the floor of the filter tank. The sand rested directly on these "strainer cups," as they were called. In spite of making the openings in the cups extremely small, more or less trouble from clogging of these openings was experienced. Better distribution of the wash water, however,

resulted since the flow was throttled in its passage through the small orifices, and entered the sand bed under a diminished but fairly uniform pressure. Wherever a manifold of pipes was employed, it was found necessary to make them of ample size so that too much head should not be lost in them by friction before the water reached the distributing nozzle farthest removed from the main supply pipe. By restricting the neck of the nozzle, so that the flow of water was throttled before it entered the small openings of the cup, even better action was obtained.

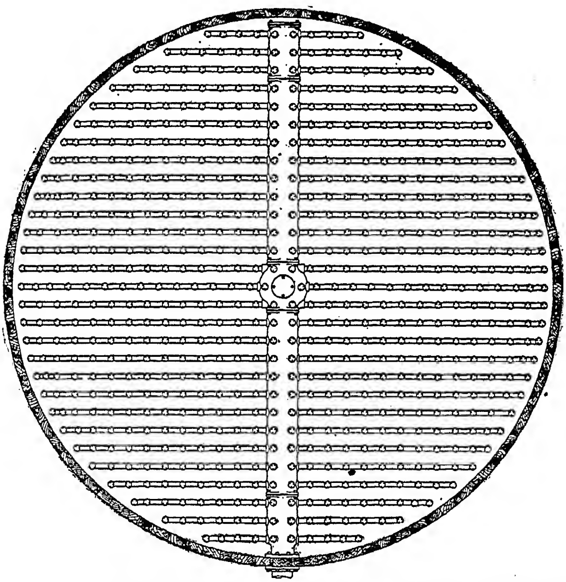

FIG. 65.—Typical arrangement of strainer nozzles and manifold for circular filter tank.

Gravel Layers.—The difficulty arising from the clogging of the strainer cup openings with minute particles of sand was met by introducing graded layers of gravel around and above the strainer system. The sand rested upon the gravel layer; and if the latter was properly graded and not disturbed in any manner, it formed an efficient barrier to the entrance of sand particles into the perforations or slots of the strainers. It then became possible to make these perforations or slots larger, and to produce a freer flow of water than could be obtained through the extremely small openings previously employed.

The principle of using gravel was not new, as the old Hyatt

cone sand valves, invented nearly 30 years ago, had made use of this idea. In these valves or strainers the screen at the top of the cone contained perforations of such size that sand could readily pass through them, but was kept from entering the system of collecting pipes by layers of shot or gravel placed between the upper and lower perforated plates of the cone valve. This shot or gravel also aided in distributing the wash water, and prevented jet action occurring to a considerable extent.

Jet action is in a measure inevitable in almost any strainer system, but the more pronounced it becomes and the further up into the filter bed the jets of water are projected, the more undesirable is the effect produced. Jets of water shot directly up through the bed wash a limited area immediately above each individual strainer. This effect is quite marked in some of the older filter strainer systems, and shows itself by depressed areas over the surface of the sand through which the greater portion of the wash water passes, leaving intermediate spaces unwashed.

Perforated Plate and Pipe Systems.—In order to more uniformly distribute the wash water, and especially to avoid the bad effect of jets of water washing limited areas of the filter bed, perforated pipes or plates (Fig. 66) extending in parallel rows either crosswise or lengthwise of the filter have been constructed. The perforated-pipe system (Figs. 67 and 68) was employed with marked success in the Harrisburg, Pa., filter plant in connection with a graded gravel bed above the pipes. At Cincinnati, Ohio, perforated plates covering concrete channels located at the bottom of trough-like depressions, which ran lengthwise of the filter, have proven entirely successful.

Hopper-shaped Filter Bottoms with Inverted Pyramidal or Prismoidal Depressions.—The perforated metal or wire-cloth bottom, theoretically at least, provided for water being discharged underneath every square inch of the gravel and sand of the filter bed. The employment of perforated brass nozzles, however, spaced some distance apart, afforded dead areas under which the water could not be applied. Gravel or sand lying between the nozzles was, therefore, imperfectly washed. To remedy this defect the shape of the bottom of the filter tank has in some instances been changed, so that the water enters at the bottom of depressions which are filled with gravel. As noted above in the Cincinnati filter bottoms, consisting of long parallel troughs with slanting sides, is found an example of this type of construction.

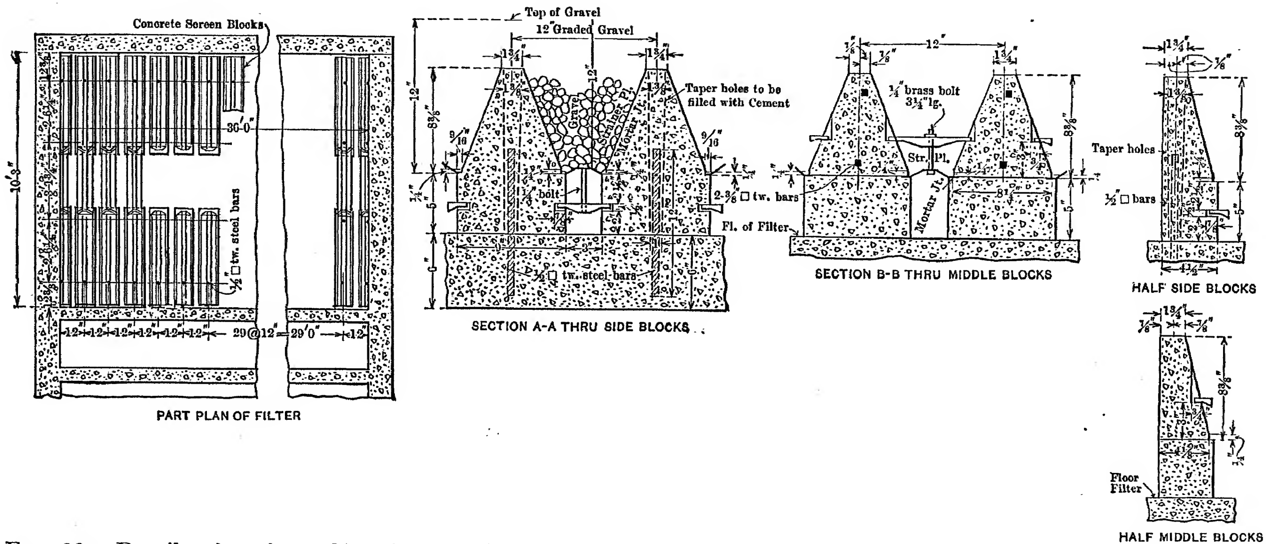

FIG. 66.—Details of perforated strainer system designed for high velocity washing at the Evanston, Ill., filter plant.

Inverted pyramidal-shaped depressions with the strainer nozzles located at the bottom of the depressions have also been used and with more or less success. The disturbance of gravel beds above such bottoms was insured against in the Cincinnati

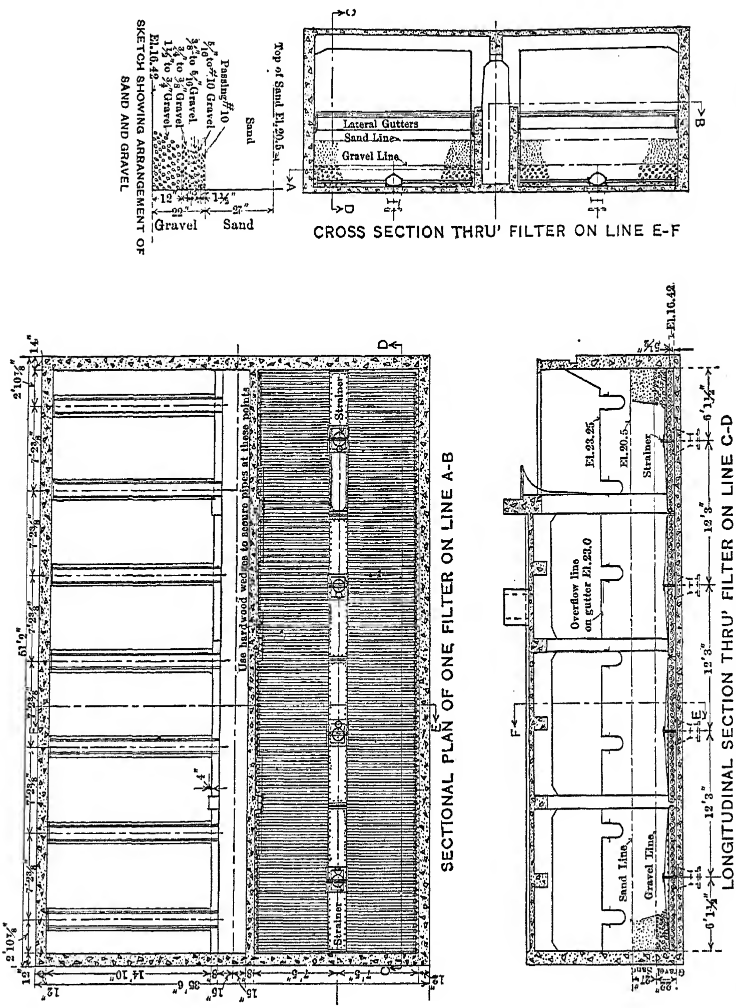

FIG. 67.—Showing arrangement of perforated pipes, gutters, and filtering material in Cleveland filter plant.

filters by the use of a screen of brass wire cloth, placed between the gravel and the sand, and which was fastened to the tops of the longitudinal ridges formed by the sides of the parallel troughs. In connection with filter bottoms containing pyramidal-shaped

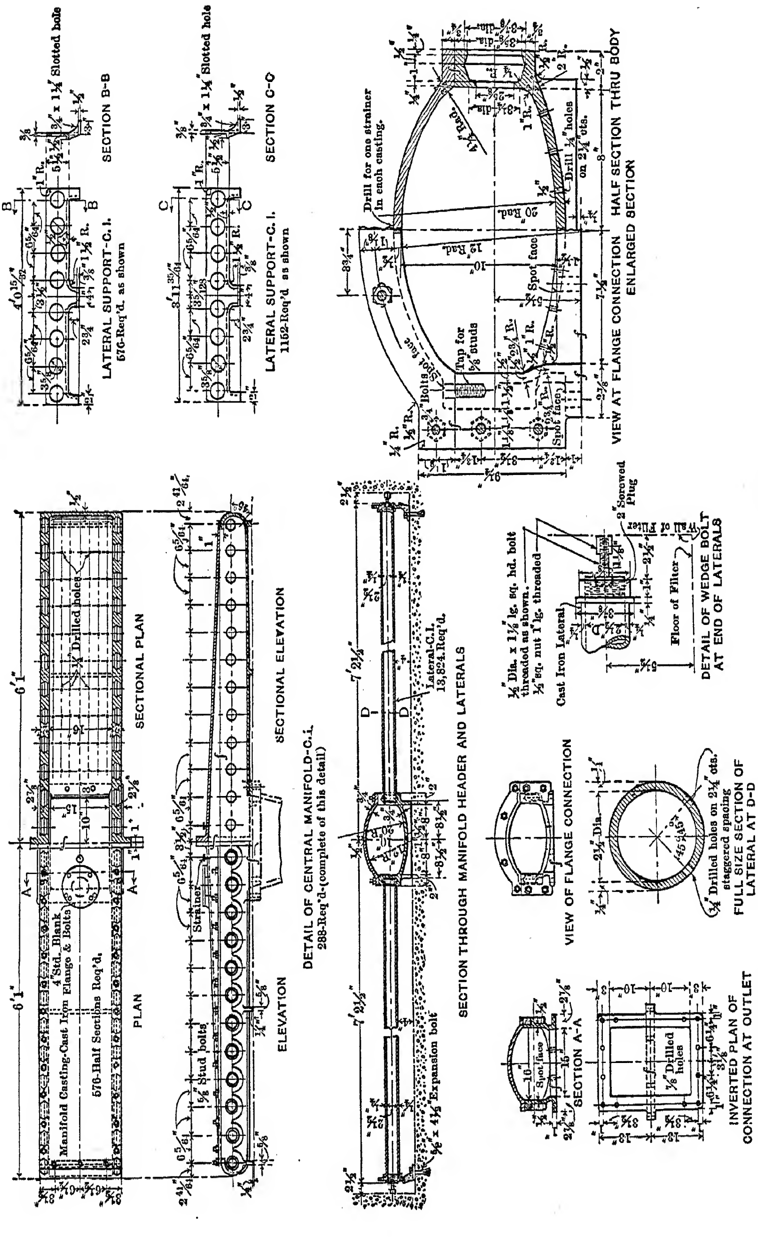

FIG. 68.—Details of perforated strainer piping and manifold header of Cleveland filter plant.

depressions, a recent design of Mr. William Wheeler of Boston, Mass., is of considerable interest, and is described below in detail.

DESCRIPTION OF TYPICAL STRAINER SYSTEMS

Strainer Systems Used at Toledo and Youngstown, Ohio, Filter Plants.—The filter tanks of the Toledo plant are 22.5 by 16 ft. by 8.5 ft. (deep), and have a nominal capacity of 1,000,000 gal. per day. The underdrain system consists of a central cast-iron rectangular manifold, into which are leaded 2-in. cast-iron laterals spaced 10 in. center to center. Slotted brass strainers of the so-called Norwood type are screwed into the laterals 6.2 in. from center to center. The most constricted part of the strainer is in the tee, which is bored out to 3/8 in. in diameter. The aggregate opening of the 837 strainers in each filter is, therefore, 92.4 sq. in., or 0.18 per cent. of the total area of the sand surface. The laterals in these filters are bedded in cement, and only the tee-shaped strainers project above the flat filter floor. This system of strainers is now being removed and replaced by a perforated-pipe system.

In the Youngstown plant the tanks are practically of the same size as those at Toledo. The main effluent manifold is a standard-weight 10-in. cast-iron pipe laid lengthwise of the tank at the center, and imbedded in the concrete floor. Transverse 1½-in. collector pipes of wrought iron placed 6 in. apart are screwed into this manifold, and are imbedded in the concrete of the floor. Into these transverse collector pipes are screwed brass strainers. The arrangement of the strainers is such that they are on 6-in. centers in each direction, making 1,176 strainers in each tank.

In many circular steel tank filters of the gravity type a false bottom plate, supported by standards and riveted to the side of the tank, is used. The brass strainer nozzles are screwed into brass-bushed holes in the plate, and are spaced about 6 in. apart. Some recent designs have made use of the false metal bottom plate provided with nozzles as above described, in rectangular filter tanks of concrete. False bottoms of concrete have been suggested, but the difficulties of designing a bottom to withstand the stresses to which it may be subjected, have not led to its adoption.

Cincinnati, Ohio, Columbus, Ohio, and Minneapolis, Minn., Filter-plant Strainer Systems.—In the filter tanks of the Cincin-

nati plant the bottoms consist of concrete channels located at the bottom of trough-like depressions running lengthwise of the tank. Each tank has 28 channels 3 in. deep, $2\frac{3}{4}$ in. wide at the top and $2\frac{1}{2}$ in. wide at the bottom, spaced on 12-in. centers between the concrete wedge-shaped ridge blocks forming the sides of the trough. The channels are connected at $12\frac{1}{2}$-ft. intervals by $3\frac{1}{2}$-in. riser pipes to a system of large cast-iron pipes under the bottom of the filter tank. The perforated plates which cover the channels are held down by nuts on the threaded end of $\frac{1}{4}$-in. hook bolts, slipped over cross-bolts spanning the channel. Strainer plates lap about $\frac{1}{4}$ in. These joints were made with red lead and the edges of the plates were cemented to the shoulders on which they rest with a Portland-cement mortar of 1 part of sand and 1 part of cement. The rise in the curvature of the plate is $\frac{5}{8}$ in. Sixty-two holes $\frac{3}{32}$ in. in diameter were drilled in each lineal foot of plate and serve a sand area of 1 sq. ft., making the strainer openings equivalent to 0.3 per cent. of the filtering surface.

In the Minneapolis plant the strainer system includes a series of waterways made of parallel ridges and grooves cast in concrete on the filter floor. The ridges instead of running lengthwise of the filter extend crosswise or perpendicular to the central wash-water gutter. They are spaced on 1-ft. centers and their tops are 13 in. above the filter floor. The channels at the bottom of these trough-shaped depressions are covered with perforated brass plates, and the area of the openings in them is 0.15 per cent. of the sand area. The filtered water collected in the channels flows inward from the sides toward the center of each half of the filter unit, and is delivered through eight 14-in. outlets in the filter floor to a manifold of 18-in. cast-iron effluent piping.

The strainer system of the Columbus, Ohio, filter plant differs from those at Cincinnati and Minneapolis in providing circular strainer plates about $2\frac{1}{2}$ in. in diameter, perforated with $\frac{1}{16}$-in. holes, and bedded in slabs of concrete placed over channels of like material built on the floor of the filter. These channels are 5 in. deep and $2\frac{1}{4}$ in. wide at the bottom. They are spaced $8\frac{3}{4}$ in. from center to center, and the perforated brass circular plates in the slabs, which cover the channels, are also spaced on $8\frac{3}{4}$-in. centers. The lateral channels connect with the eight main collectors formed in the concrete bottom of the tank, which in turn

connect through iron castings set in the filter floor with piping below the tank.[1]

Harrisburg, Pa., Filter-plant Strainer System.—In this plant is found a very simple strainer system (Fig. 69) which was based on the idea that if a gravel underdrain system could be prepared that would hold the sand up under the actual conditions of operation, strainer cups of any sort were unnecessary, and underdrainage piping could be considerably simplified. The system

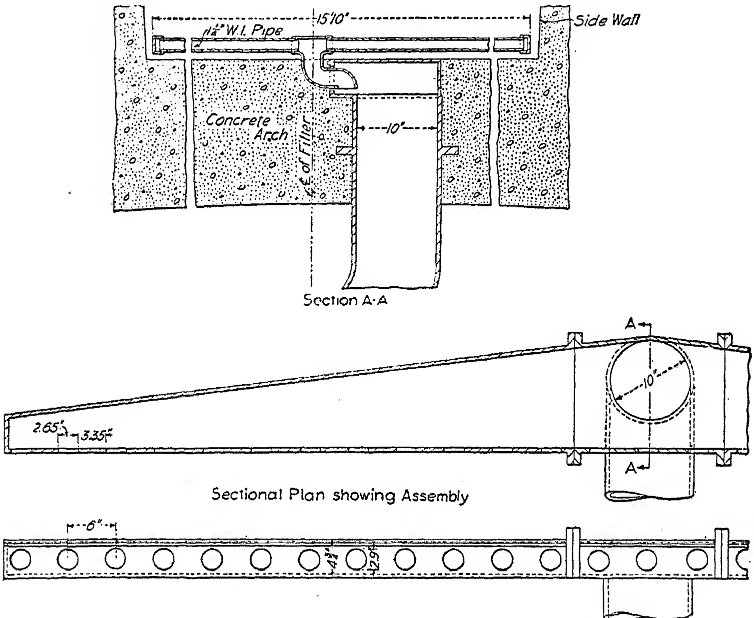

FIG. 69.—Details of special castings and pipe in underdrain system of Harrisburg, Pa., filter plant.

as constructed consisted of a series of parallel lines of 1¼-in. galvanized-iron pipe laid 6 in. apart and running crosswise of the filter, the pipes being drilled along their under surface with a row of holes 7⁄32 in. in diameter and 3 in. apart from center to center. The pipes were capped at their outer ends, and at the center entered a tee of special form, which connected with the side of a cast-iron manifold laid in the concrete floor of the filter. The manifold was connected with the main effluent pipe of the filter.

[1] *Eng. Record,* vol. 53, Feb. 24, 1906.

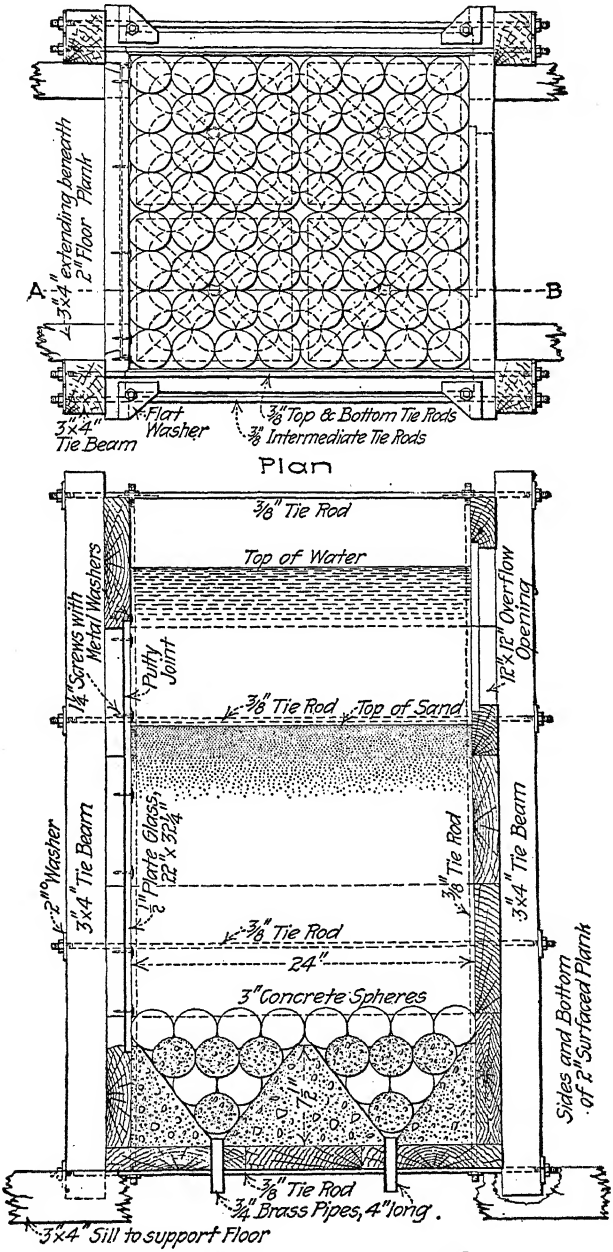

FIG. 70.—The Wheeler bottom for rapid sand filters.

Belfast, Me., and Akron, Ohio, Filter-plant Strainer Systems.—There has been recently developed a form of strainer bottom which departs materially from the general principles involved in the systems previously described. This system was invented by Mr. William Wheeler of Boston, Mass., and as described (Fig. 70) by Mr. R. S. Weston[1] consists of inverted truncated pyramids placed 1 ft. apart on centers. They are not deep enough to meet at their sides and form sharp edges. Each pyramid is provided with a 3/4-in. outlet. Over each outlet is placed one sphere 3 in. in diameter, made of neat Portland cement, and above this a layer of four similar spheres. Above the four spheres are placed nine blue glazed earthenware marbles, eight of which are about 1 1/4 in. in diameter, and one (at the center) about 1 9/16 in. in diameter. Above the marbles are placed 6 in. of graded gravel all of which has passed through a 1-in. ring. In other words, the underdrain system consists of a system of spheres arranged like an inverted pile of cannon balls, with its center ball directly over the outlet pipe.

The sphere directly over the outlet produces the so-called "ball nozzle" effect, and the tendency of the water to adhere to the sides and surfaces of the balls is that which produces so uniform a distribution of the incoming wash water in the washing process. This distribution begins near the point of entrance, and it is the uniform arrangement of the balls, as well as the adhesion of the water to their surfaces and to the sides of the pyramid, that perfects the distribution before the sand layer is reached. This system will be discussed again in the description of methods of washing.

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[1] *Eng. News*, July 2, 1914, p. 22.

CHAPTER XVIII

DETAILS OF RAPID SAND FILTER-PLANT CONSTRUCTION (CONTINUED)

DEVICES FOR USE IN AGITATING THE FILTER BED DURING THE WASHING PROCESS

In the early development of rapid sand filters the proper cleansing of the sand bed was soon recognized as of the utmost importance. The forcing of water through the filtering medium in a reverse direction from that during filtration cleared the sand bed of a great deal of its accumulated sediment, but did not always do it as thoroughly as was desired. The upward rate of flow of the wash water ranged from 8 to 12 in. per minute, and caused some slight expansion in the volume of the sand bed. In order to be certain that wash water had access to all parts of the bed, and that no sand escaped the cleansing action of the water, stirring devices were placed in the bed and were rotated during the application of the wash water.

Mechanical Rakes.—As the earlier rapid sand filters consisted of cylindrical tanks, rotating rakes for agitating the sand bed in the washing process were found to be quite satisfactory. Since the filter bed consisted entirely of sand, it was possible to stir the bed to practically its full depth. The agitating devices usually consisted of a set of rake arms, each arm being provided with several teeth. The rake arms were hung from a vertical shaft, which was provided at its upper end with a horizontal gear, engaging either a worm or bevel gear.

A small steam engine usually provided the power which it transmitted to a driving pulley. By means of belts and pulleys or gears, the movement of the vertical-shaft gears mentioned above was effected. Devices were also employed which enabled the rakes to be lowered into the bed as they rotated, and to be raised again just before the wash water ceased to flow up through the bed.

The teeth of the rake arms were from 3 to 4 ft. in length, and they revolved at a rate of eight or nine times a minute. The

teeth descended into the bed to within 2 or 3 in. of the bottom. The teeth were square or wedge-shaped in section, and were spaced about 6 in. apart. Sometimes shorter teeth were used, on the end of which were attached chains. These chains were long enough to drag on the sand and were used for surface agitation

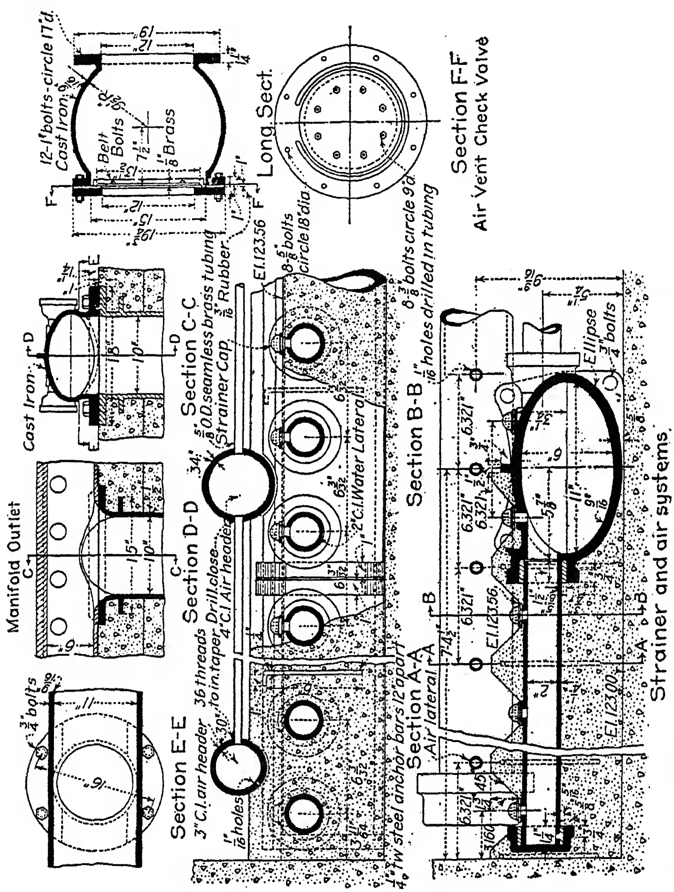

FIG. 71.—Details of air and water systems designed for the Jerome Park filter plant, New York.

of the sand bed when it was not desired to agitate the sand by driving the rake teeth into the bed.

Systems of Air Agitation.—With the advent of concrete tanks for filter purposes, which could be built so much more economically in a rectangular form, and which could be made so much larger than the circular wooden or steel tanks, the rotating mechanical agitating devices were soon found to be unfitted for use in them,

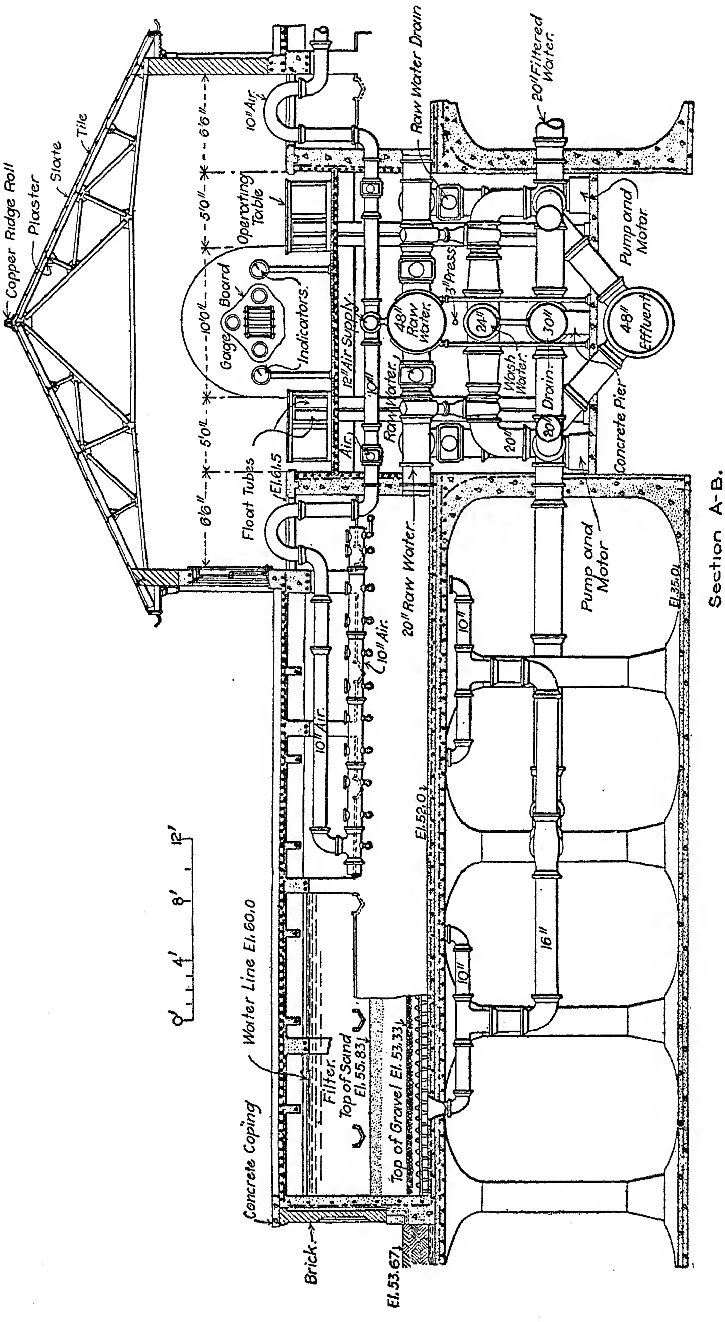

FIG. 72.—Cross-section of Columbus, Ohio, filters showing air piping and water piping.

as they were cumbersome and awkward to operate. The use of air under low pressure was suggested as offering a means of agitating the filter bed, and its employment soon became quite general.

Two general methods are in vogue for distributing air under pressure to filters for agitating purposes. In one method the air is supplied through a separate system of pipes and exits to the bottom of the sand bed (Fig. 71), and in the other the main supply pipes deliver the air to the underdrain system of water pipes of the filter, from which it escapes through the strainers into the filter bed. Because of the disturbing effect of the ascending air on the finer gravel layers of the filter bed, the air-delivery piping system is sometimes placed between the gravel and sand layers. Obviously, this can not be done if the water strainer system also serves as an air-delivery system.

The main air-supply pipe coming from the blower to the filters is usually of cast iron. Branch distributors pass over the tops of the filter tanks and then downward, connecting with the independent distribution piping (Fig. 72), or with the water-strainer system, when the latter serves the double purpose of distributing both air and water as described above. Cast-iron piping is usually employed for air distribution up to the laterals both in the separate and the combined systems. In the separate system the laterals are usually seamless brass tubing perforated with openings of 1/16 in. diameter, and spaced about 6 in. apart.

Air is usually forced into the bed at the rate of 3 to 5 cu. ft. per minute of free air per square foot of filter surface. Usually the application of air precedes the application of the water, but in some cases the two are applied together.

Depth of Sand and Gravel.—In the earlier rapid sand filters when sand alone was used in the filter bed, its depth ranged from 30 to 50 in. Usually more or less sand was lost in washing, depending upon the violence of the agitation with the mechanical rakes and the upward velocity of the wash water. The shorter the distance between the normal level of the sand and the edge of the waste-water troughs, the greater the amount of sand lost during the washing process. Consequently, variations in design and methods of operation resulted in considerable differences in the actual depth of the sand bed. Renewals of the sand bed were more or less frequent, and a bed which had originally a depth of 4 ft. might decrease to 2½ ft. before more new sand was placed in the tank.

Present practice usually provides a depth of sand ranging from 30 to 36 in. Depths less than 30 in. may be used successfully if the sand is fine enough. Deeper layers are usually provided where the sand is relatively coarse.

The size of the sand particles is of importance if good purification is to be obtained. Sand with an effective size of only 0.26 mm., and a uniformity coefficient of 1.68 is being used in a 30-in. layer in a plant recently constructed. Sands having an effective size of 0.5 to 0.6 mm. are frequently used, and as they offer much less frictional resistance to the flow of water, are probably efficient with certain classes of waters. Coarse sand was quite commonly employed in the earlier filters. Later, the use of a finer sand came into vogue. Present practice appears to be again swinging to the use of the coarser sands. With the early filters the requirements as regards clarification and bacterial removal were not as rigid as they became later when they began to be used for the purification of municipal water supplies. The use of sterilization agents of late years has again relieved the filters of such exacting demands in the way of bacterial purification, and provided the sand is fine enough to properly clarify the water, it is usually suitable to effect the degree of bacterial removal demanded in conjunction with disinfecting agents.

The proportion of very fine material in a sand should not be too high. For example, it is sometimes specified in buying sand that not more than 1 per cent. shall be finer than 0.25 mm. Where it is possible to wash the sand bed at a high rate after it is placed, the amount of fine material need not be given very much consideration, as it is very easy to wash it out by repeated washings, assisted by a light scraping of the sand surface between washings. The hydraulic grading of the bed by this method of washing makes this separation of the fine material easy and economical.

Gravel layers vary in depth, but usually range from 6 to 10 in. Much deeper gravel layers are used where certain methods of filter washing are in operation, and as high as 18-in. layers have been employed in some instances. For high-velocity methods of washing probably 14-in. layers are more commonly used, since this appears to be about the depth required to suppress the jet action of the rising wash water. Gravel layers are usually graded, and the sizes of the stones vary from 3 in. in diameter in the bottom layers to $\frac{1}{16}$ in. in diameter in the top layers.

The following table gives the depths and sizes of sand and gravel used in four modern rapid sand filter plants:

DEPTHS AND SIZES OF SAND AND GRAVEL USED IN VARIOUS RAPID SAND FILTER PLANTS

	Filter plants at			
	Cincinnati, Ohio	Harrisburg, Pa	Columbus, Ohio	Columbus, Ind.
Depth of sand	30 in.	30 in.	30 in.	30 in.
Effective size	0.34 mm.	0.39 mm.	0.41 mm.	0.35 mm.
Uniformity coefficient	1 60	1 40	1 36	1.60
With or without wire screen between gravel and sand	With	Without	Without	Without
Depth of gravel	9½ in.	7 in.	10 in.	18 in.
Sizes of particles, inches	Depths of the various layers, inches			
1½ to 3	—	—	—	10
1 to 1½	—	—	—	4
½ to 1	1	—	2	—
¼ to ¾	—	4	—	—
¼ to ½	7	—	2	3
⅛ to ¼	½	—	3	—
1/12 to ¼	—	3	—	1
1/12 to ⅛	1	—	—	—
1/16 to ⅛	—	—	3	—

Screens between the Gravel and Sand Layers.—The disturbance of the gravel layers of a filter is obviously undesirable, if dependence is placed upon them to maintain an efficient barrier between the sand bed and the strainer system. As a support for the sand bed, or as a distributor of wash water, the gravel bed must at all times remain properly graded. The partial "inversion of the bed," that is the formation of pockets of sand or of gravel reaching from the surface of the filter bed to the strainer system, may produce poor purification results. This condition of the filter bed has been frequently found. Filter beds thus disturbed could not be properly washed, since differences in frictional resistance to the flow of water between the disturbed and undisturbed portions of the sand and gravel layers made uniform cleansing of them impossible. For the same reason uniform rates of filtration could not be maintained over the whole area of the filter bed.

The cause for the initial disturbance of the gravel bed was usually found in the strainer system, which produced such a pronounced jet action at certain points in the bed as to gradually displace the gravel and permit the sand to drop back into the space thus produced. The injection of compressed air into a filter bed for the purpose of agitating the sand layer also assisted in displacing the gravel, especially when the air entered through the water-strainer system below the gravel bed. Injection of air and water together caused even greater displacement effects than air alone.

To remedy these difficulties, the use of a brass wire-cloth screen placed and rigidly held between the gravel and sand layers was tried. The results obtained were entirely satisfactory, since jet action of the strainers was minimized, and since the gravel bed was maintained in the original condition in which it was laid. The use of the screen also led to another interesting development in the method of washing a filter, now known as the "high-velocity method," and which will be described later under methods of operation.

The screen was first used in the Cincinnati filter plant, and resulted from extended hydraulic experiments undertaken prior to the design of this plant. In constructing the filter beds of this plant the gravel was first carefully placed in the longitudinal troughs, filling the latter to the top. The brass wire cloth was then laid on the surface of the gravel layer and fastened to the tops of the ridges with nuts and washers on brass bolts, which latter had been previously set in the tops of the ridges when they were molded. The bolts were ¼ in. in diameter and were spaced 18 in. center to center. The wire cloth had been woven into rolls 50 ft. long and 26 in. wide. It was made with 100 meshes per square inch, and the wires were of size No. 20, "Old English gage." The brass contained 25 per cent. of zinc and 75 per cent. of copper.

The 26-in. widths of wire cloth extended across two troughs or depressions. Where two widths lapped, the edges were further fastened by driving double-pointed copper tacks through the two layers. The points of the tacks bent over after passing through the cloth by striking the concrete, and thus clipped the edges together. The methods of fastening down the screen proved to be inadequate after a year or so of operation. At other large plants where the screen is used, better methods of fastening have

been employed and no trouble from tearing away of the screen from the bolts and fastenings at the seams has been experienced.

The life of the brass wire cloth has proven to be very short with certain waters. At Cincinnati and Louisville, Ky., the brass wire cloth in the filters has corroded badly and has finally become worthless. At Grand Rapids, Mich., and at New Orleans, La., on the other hand, the screen has not corroded and is still in excellent condition. In both of the two last-mentioned filter plants the method of treating the water produces a water having practically neither free nor half-bound carbonic acid, while at the two plants where the screen has corroded badly, the method of treating the water is such that in one case no free CO_2 is present, but half-bound carbonic acid is always present (Cincinnati), and in the other both free CO_2 and half-bound carbonic acid are always present (Louisville). It would seem as though the corrosive action of the water was due to the small amount of half-bound carbonic acid present.

It has been found possible to avoid the use of the brass wire cloth by deepening the gravel layers, and by making them of somewhat larger material at the bottom. This phase of the design will be further discussed under methods of operation.

Velocity of Flow of Water through Sand Strainers and Underdrains.—In the design of the Jerome Park filters for the purification of the Croton water supply of New York City, several types of strainer systems were worked out. Mr. George W. Fuller[1] has briefly described these designs and given a table showing the working velocities for the flow of the water during filtration and filter washing for the different types of design.

"Type A consisted of cast-iron manifolds, cast-iron laterals with brass strainer cups for washing at the rate of 9 gal. per minute per square foot of sand surface, and a separate air system for air wash through cast-iron manifolds, and brass laterals, at the rate of 4 cu. ft. per minute per square foot of sand surface.

"Type B provided for the application of both air and water through the same system, using the Williamson strainer. This strainer is a cup screwed into the top of the pipe with an elongated neck projecting downward to the bottom of the pipe.

"Type C consisted of concrete channels and perforated brass plates for applying water only at the rate of 15 gal. per minute per square foot of sand surface. The sand and gravel were separated by a brass wire screen.

[1] George W. Fuller: "The Croton Water Supply: Its Quality and Purification." *Jour. Am. Water-works Assn.*, March, 1914.

"In the following table are given the working velocities through the various parts of the three strainer systems for filtration and washing."

JEROME PARK FILTERS. STRAINER AND AIR SYSTEMS—WORKING VELOCITIES, FEET PER SECOND

	Filtration	Washing	Air (free)
Type A:			
Through sand	0 00443	0.02	0 0668
Strainer holes or air holes	1.50	6 75	260 0
Strainer neck	2.20	10.00	
Laterals	0 71	3.18	86 0
Manifold	1.11	5 00	79 0
10-in. supply	1.46	6 60	
18-in. collector	1.80	8 15	
24-in. main	2 03	9 17	
16-in. air main	....	...	68.8
Type B:			
Through sand	0.00443	0 02	0.0668
Strainer holes	1.55	7 10	23 40
Strainer neck	2.15	9.70	360.00
Laterals	0.58	2.60	8.68
Manifold	1.11	5 00	8 32
10-in. supply	1.46	6.60	
18-in. collector	1.80	8.15	
24-in. main	2.03	9.17	
16-in. air main	....	...	68.80
Type C:			
Through sand	0 00443	0.0333	
Strainer holes	1.41	10.80	
Laterals	0.26	1.98	
Manifold channel	0.40	3.08	
14-in. supply	0.73	5.60	
24-in. collector	0.99	7.61	
24-in. main	2.00	15.20	

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2. GEORGE W. FULLER: "Report on Water Purification at Cincinnati, Ohio."
3. *Eng. Record*, vol. 63, p. 65.
4. *Eng. Record*, vol. 64, p. 379.
5. *Eng. Record*, vol. 69, p. 529.
6. GEORGE W. FULLER: "Some Features of Detail in the Design of Rapid Sand Water Filtration Plants." *Engineering-Contracting*, Sept. 2, 1914.
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CHAPTER XIX

REGULATING, MEASURING AND INDICATING DEVICES FOR RAPID SAND FILTER PLANTS

RATE-OF-FLOW CONTROLLING APPARATUS

Apparatus for controlling the rate of flow of water through rapid sand filters are now usually found in all modern and well-equipped plants. Sudden variations in the rate of filtration, especially where the change is from a lower to a higher rate, are likely to produce poor purification results. If, instead of a fixed rate of discharge for the whole period of service of the filter, a changeable rate of flow which meets variations in the consumption of filtered water, is desired, it is well that the change of rate should be gradual so as not to subject the filter bed to suddenly increased or decreased pressures. It can be easily understood that a suddenly increased pressure might cause the bed to break through. It is also conceivable that rapidly decreasing the rate of flow might be injurious, since air which may be held in the bed at the higher rate might be liberated at the lower. This air would escape by breaking through the surface film and produce in the beds points of lessened frictional resistance, through which poorly purified water could escape to the underdrains.

The rates of flow generally permitted through rapid sand filterr vary considerably. They may range from 75,000,000 to 150,000,000 gal. per acre per day, although the usual rates are from 100,000,000 to 125,000,000 gal. per acre per day. As the initial minimum frictional resistance to the flow of water through a clean filter at the usual rates of filtration will vary from 1 to 3 ft., and the maximum resistance of the dirty filter from 10 to 12 ft., it is evident that controlling apparatus must be constructed so as to operate within these limits, either to produce a uniform or variable rate of flow as may be desired.

The loss of head through the passages of the rate controller at its normal capacity should be low, that is from 6 to 9 in. Surging of the water through the controller indicates a lack of sensitiveness to changing rates of flow and is undesirable. The

best designed controllers will maintain the rate of flow within 2 or 3 per cent. of the mean rate for which the controller is set. Means for adjusting the rate by changing the size of the orifice through which the water is controlled are usually provided in all well-equipped machines of the orifice type. In the Venturi type of controller this adjustment of rate is taken care of in various ways as will be shown in the description of typical machines.

Rate controllers may be calibrated by noting and marking the sizes of the openings in the measuring or controlling orifices when the volume of water flowing through them is that desired. If facilities for measuring the volume of water discharged through these orifices for various settings of the controller are at hand, this calibration may be done before the controller is set in place If this is not possible, the filter tank itself may be utilized by noting the lowering of the water in the tank with the influent valve closed, and calculating from the size of the filter tank the volume of water flowing through the controller for any given length of time.

A Simple Method of Rate Control.—The rate controller is placed in the main discharge pipe of the filter between the filter tank and the filtered water effluent pipe which collects the water from all the filters. The depth of water over the sand bed is usually maintained fairly constant. This latter condition is not complied with, however, in a very simple form of control recently designed.[1] This consists of a disk inserted in the discharge outlet of the filtered water pipe. This disk has an orifice of such a size that the rate of flow desired will be at the maximum rated capacity of the filter when the sand is clean and the head of water above the sand at some minimum level. If another orifice plate under a constant head of water is placed in the influent pipe of the filter, the delivery of water to the latter can be maintained at the same rate as the discharge. As the filter becomes clogged the height of the water above the sand will increase, thereby increasing the head on the outlet orifice and tending to maintain a practically constant discharge. By this method a positive pressure is maintained at all times on the outlet orifice. The gradually rising level of the water above the sand becomes a direct measure of the frictional resistance produced by the clogging of the sand. When the water above the sand has reached

[1] "Revamped Water Works and New Purification Plant at Fort Smith, Ark." *Eng. Record,* Sept. 5, 1914, p. 262.

its maximum level, the filter must, of course, be stopped and washed, if the rate of flow is to be maintained.

The above principle of control is not the most common one in use. The rate at which water will pass through a clean sand bed and through the usual system of underdrains, filter effluent piping and valves is far in excess of the safe rated capacity for rapid sand filters. It, therefore, is necessary to restrict the rate of discharge by introducing some frictional resistance at the outlet of the filter. This is easily done when the filter is placed in service by partially closing the effluent valve until the maximum rate desired is produced. By opening this valve a little at a time during the period of service of the filter, the increasing frictional resistance in the sand bed, due to clogging of the interstices of the sand layers, may be compensated for by reducing the frictional resistance through the outlet valve. When the latter is wide open the rate can no longer be maintained if the head lost in the sand bed reduces the effective pressure below that required to produce the desired rate of flow. It is at this point that washing the sand must take place if the rated capacity of the filter is to be maintained. However, it is sometimes the case that operators prefer to obtain as much filtered water as possible from a filter between cleanings, and, therefore, permit the filter to operate at a decreasing rate of discharge until the volume of water obtained becomes too small to make it worth while to continue filtering.

Automatic Rate Control.—The proper hand regulation of the effluent valve is obviously dependent upon the skill and attention of the attendant. It can be much better effected by automatic control. There are quite a number of automatic devices now in use which successfully control the throttling of the effluent valve itself, or more commonly of a specially balanced valve which is more sensitive to the action of the auxiliary apparatus required to move the valve. The auxiliary apparatus usually depend for their action either upon a float operating in an open chamber (Fig. 73) and moving directly various types of valves which control the flow of water from the filter; or upon changes in the difference in pressure upon two sides of a fixed orifice, which force is directly or indirectly employed to actuate valves controlling the flow; or upon changes in the difference in pressure in the full and constricted portions of a Venturi tube (Fig. 74), likewise utilized to directly or indirectly operate a valve controlling the outflow of

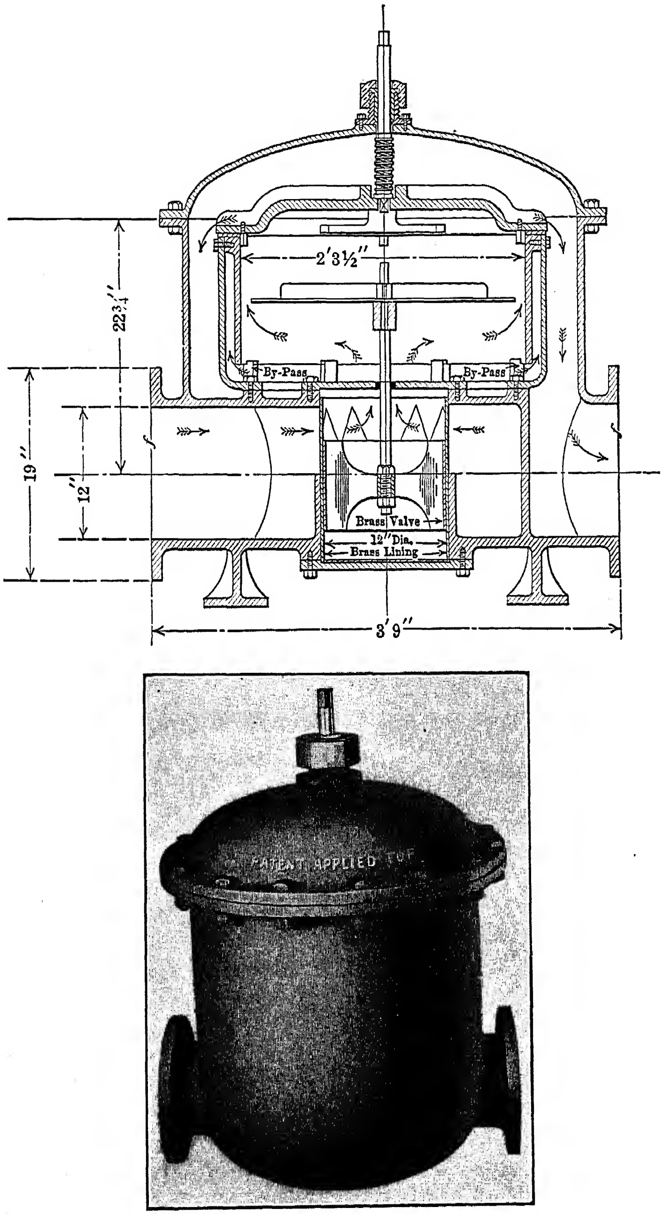

FIG. 73.—Rate controller operated by float of disk type.

water from the filter. In the float type of controller a movable weir is generally used. This type of controller can not be operated submerged. The other types of controllers, as a rule, transmit the slight differences in pressure produced by the orifice or Venturi tube to auxiliary devices, which in turn make use of electrical or hydraulic power to actually move the controlling valve. This class of controllers may be operated submerged. The manner in which these devices operate can be more easily understood by a description of some of the typical rate controllers now in use.

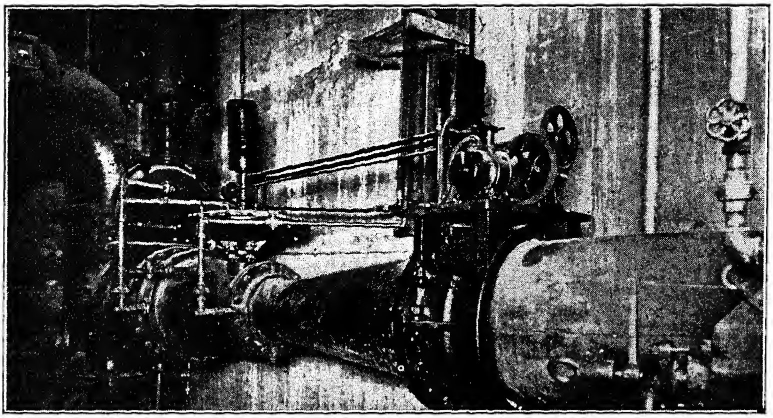

Fig. 74.—Venturi type of rate controller.

Weston Rate Controller.—One of the earliest rapid sand filter controllers to be used was that invented by Mr. E B. Weston[1] (Fig. 75). This controller, as may be seen from the.cut, is of the open type, and consists of a chamber containing a float which is attached by levers to butterfly valves, and which admit the water from the inlet pipe to the controller. The float in the controller body is mounted on a hollow stem, and moves in guides located at the top and bottom of the controller casing. Beneath the float is a deflector designed to quiet the water and to diminish the effect of currents, thereby giving a smooth entrance to the discharge tube, and being aided in this respect by the flaring ring at the mouth of this tube. Mounted also on the float stem at a fixed distance below it, so as to be maintained at a constant depth, is a disk, which is turned with a thin edge and sharp corners, and

[1] *Eng. Record,* Nov. 25, 1899, p. 596.

of such diameter as will give the annular orifice between the disk and the walls of the discharge tube a predetermined area proportional to the desired rate of discharge. As the float is mounted on the stem at a fixed distance from the disk, and as the float is free to move up and down with the water, a constant head of water is, thereby, maintained above the annular orifice formed by the disk and walls of the discharge tube. The rising float tends to close the butterfly valves and thus decrease the inflow to the controller from the filter and *vice versa*.

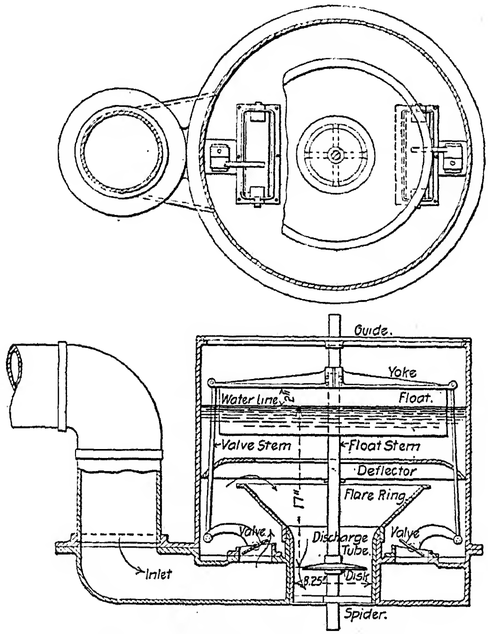

Fig. 75.—Weston rate controller.

Controllers at the Hackensack Filter Plant.—A somewhat later design of controller than that of E. B. Weston's was installed at the Hackensack filter plant (Fig. 76). This controller is also of the open type and is operated by a float which moves a balanced inlet valve. Each controller was set in a concrete-steel tank 6 by 6 ft. by 5 ft. 10 in. (deep) with its outer end rounded, and with its flattened inner end against the inner wall. The top of the tank was 2 ft. above the high-water level in the filtered-water reservoir. The inlet to the controller from the filter was at the

bottom of the former through a 20-in. pipe casting. To the upper end of the pipe casting was bolted the body casting of the balanced valve, and to the upper end of the latter was attached the body of the controller. This portion of the device consists of an outlet pipe and an adjustable cylindrical brass weir, connected above to a float and below to the balanced valve previously mentioned.

The balanced valve body has two sets of openings, one above the other, separated by a metal barrel 3 in. high. In each set are

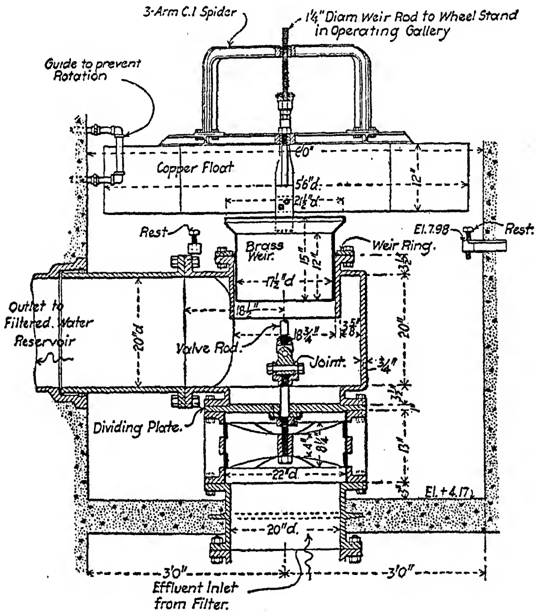

FIG. 76.—Rate controller used at the Hackensack, N. J., filter plant.

six openings 2½ in. high, separated only by narrow vertical bars of the metal. The valve cylinder has but one set of openings at its middle. These openings are 2¾ in. high. The cylinder is so attached to the valve rod that when it is in its central position all the annular openings just mentioned are closed. The extreme travel of the cylinder is 2½ in. vertically up or down, and is operated as previously stated by the float in the chamber above.

A brass cylinder with a flaring top 20½ in. in diameter is attached to the stem connecting the float and balanced valve, and forms the weir over which the water passes to the outlet of the

controller. By means of a hand wheel on a stem extending from the weir cylinder to the operating floor, the distance between the float and weir edge may be adjusted, and thereby fix the head of water upon the weir, and consequently the rate of filtration. When adjusted for any given rate the rising and falling of the float automatically decreases or increases the amount of water admitted through the balanced valve, and hence maintains the rate of flow for which the weir is set.

Vivian Rate Controller.—In connection with the construction of the Cincinnati filtration plant a rate controller was designed

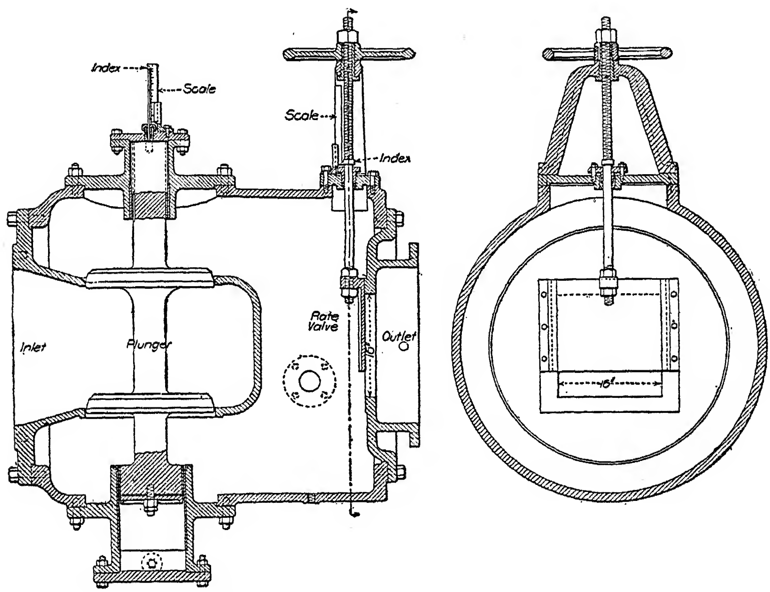

FIG. 77.—Sectional view of Vivian rate controller.

by Mr. Simon Vivian, which in practice has given very satisfactory results. This controller is of the submerged type, and consists of a balanced valve (Fig. 77) moved by hydraulic pressure. Its distinctive feature is its positive control of the balanced valve by an external source of hydraulic pressure. This external pressure is applied and released by the change in the difference in head of the water, whose flow is being controlled, upon the up- and downstream sides of an orifice, the size of which may be adjusted by means of a gate. This control is effected through the medium of an auxiliary valve (Fig. 78), hung from a flexible rubber-

cloth diaphragm, whose movement upward or downward is brought about through the above-mentioned change in the difference in head upon the up- and downstream sides of the orifice. The diaphragm and pendant valve are located without the main body of the controller in a separate casting.

By reference to the cut (Fig. 79) it will be seen that the main body of the controller containing the balanced valve is attached directly to the effluent pipe of the filter. The water entering the controller strikes the dead end of the inner casting and is diverted upward and downward through ports in which

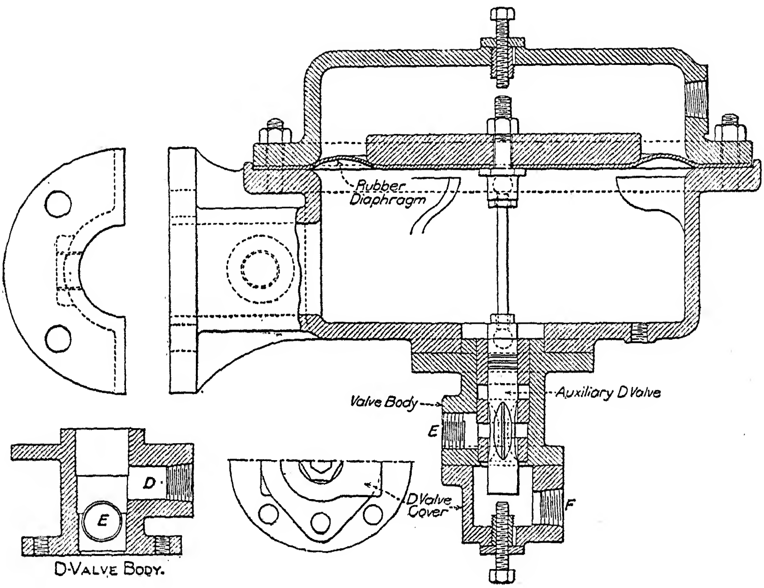

Fig. 78.—Sectional view of auxiliary valve of Vivian rate controller.

are moving the disks of the plunger throttling valve. The latter consists of a shaft to which are attached two circular disks, so placed on the shaft as to practically close the ports when entering the latter. The water after passing the ports enters the outer body of the controller and flows out through the adjustable rate valve or gate.

The plunger throttling valve is moved upward by an external source of hydraulic pressure exerted within the cylinder at the bottom of the controller body, and acts on the lower end of the plunger shaft to move it upward. When the pressure is released,

the plunger falls of its own weight. The cylinder at the top is merely a guide box to insure perfect alignment of the shaft.

The application and release of this hydraulic pressure is brought about by any change in the difference in pressure upon the up- and downstream sides of the rate gate, and is communicated by pipes to a small cast-iron box divided into two parts by a flexible rubber-cloth diaphragm. Pendant from the diaphragm is a cylindrical valve provided with D-shaped ports. This valve is free to move up and down through a bronze-lined casting attached

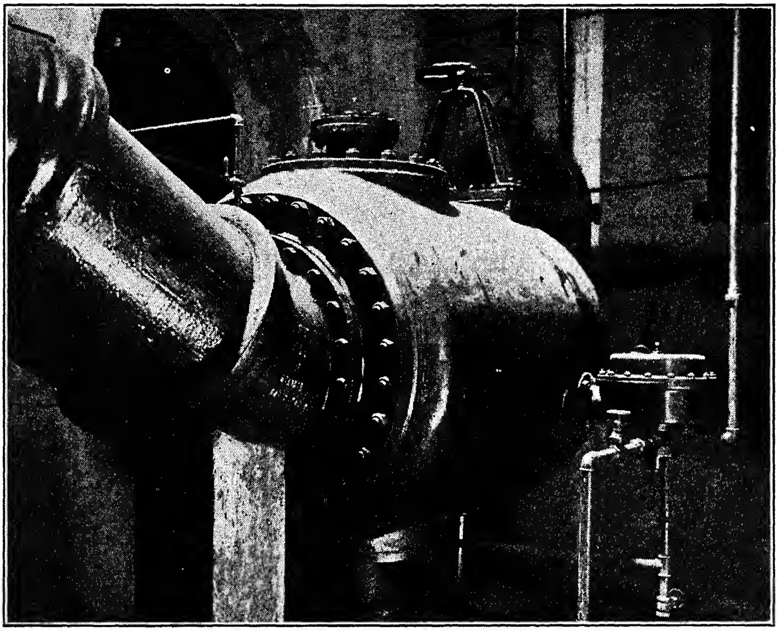

FIG. 79.—Vivian rate controller, outside view.

to the bottom of the diaphragm box. This valve box is provided with three pipe openings. One opening is connected to the source of hydraulic pressure, one with the bottom of the pressure cylinder, in which moves the end of the plunger throttling valve, and one with a waste or drain. At certain positions of the diaphragm, the auxiliary-valve ports permit water to pass from the pressure pipe to the cylinder, and thus raises the plunger of the throttling valve and cuts off or reduces the flow through the controller; at the other extreme position the communication from the pressure line is cut off, and that with the cylinder is opened through the ports

of the auxiliary valve to the waste pipe, whereby the pressure in the cylinder is released and the plunger throttling valve drops.

This movement of the auxiliary valve depends upon the movement of the flexible diaphragm, the lower side of which is open to the pressure of the water in the body of the controller above the rate gate or orifice, and the upper side of which is open to the pressure of the water on the downstream side of the rate gate. A balance is thus effected, which tends to maintain, for any given opening of the rate gate, a constant difference in pressure between its up- and downstream sides. The diaphragm virtually floats in the water and tends to assume a neutral position, which by neither letting in water under pressure to the under side of the plunger shaft, nor letting it out, holds the latter in a practically fixed position for any given head acting through the inlet of the controller. A change in this head causes a readjustment of the position of the plunger.

For example, an increase in the head above the rate gate raises the diaphragm and in consequence the plunger of the throttling valve, thus checking the flow and decreasing the head. On the other hand, an increase in the head below the rate gate depresses the diaphragm and releases the pressure of the water in the cylinder of the throttling valve, and causes it to drop, thereby decreasing the loss of head and again tending to establish a constant difference in head for any given opening of the rate gate. This is, of course, the condition necessary for a constant rate of flow.

The auxiliary valve of this controller has been effectively employed in connection with a hydraulic valve in place of the balanced valve, and using an orifice plate instead of a rate gate. This of course decreases the first cost of the apparatus, since the hydraulic valve may be used both as the effluent valve of the filter and the throttling valve to control the flow.

Simplex Rate Controller.—The type of controller which depends upon changes in the difference in pressure in the full and contracted sections of a Venturi tube is illustrated by the Simplex rate controller (Figs. 80 and 81). In this apparatus a Venturi tube forms part of the effluent pipe of the filter. On its downstream side is placed a balanced valve. This valve, as shown in the cut, consists of an outer and inner casting; a balanced valve, which moves up and down in the inner casting; annular ports for the escape of the water from the outer to the inner casting; a weighted arm attached to the stem of the valve in order to

counterbalance its weight; and a flexible diaphragm attached to the lower end of the stem of the balanced valve. The diaphragm forms a closed chamber at the bottom of the outer casting. This chamber is connected by a pipe with the throat of the Venturi tube. The upper side of the diaphragm is open to the pressure of the water in the outer casting.

The weight upon the arm attached to the stem of the balanced valve is movable, and by setting it in the proper position for the rate of flow desired will maintain the correct port openings in the balanced valve. An increased velocity through the constricted portion of the Venturi tube decreases the pressure at this point. This diminished pressure is transmitted through the pipe to the

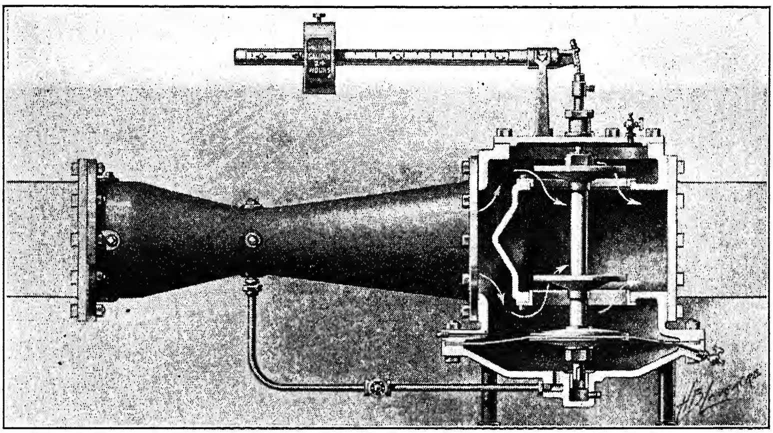

FIG. 80.—Simplex filter rate controller.

under side of the diaphragm. For any balanced position of the valve this diminished pressure underneath the diaphragm would cause it to lower slightly, and thus pull down the balanced valve and partially close off the annular ports through which the water is escaping. By throttling the outflow in this manner, the velocity through the Venturi tube would be diminished, and the pressure at the throat increased. This increased pressure, being transmitted to the under side of the diaphragm, would have the effect of moving it back to its original position. In this manner the size of the orifice is adjusted automatically to maintaining the desired rate of flow from the filter.

Earl Rate Controller.—This controller, invented by Mr. George G. Earl, depends for its action upon the difference in pressures

transmitted by a piezometer and Pitot tube, which are located in the constricted throat of a pipe forming part of the effluent main of the filter; or it can as well be used with any other pres-

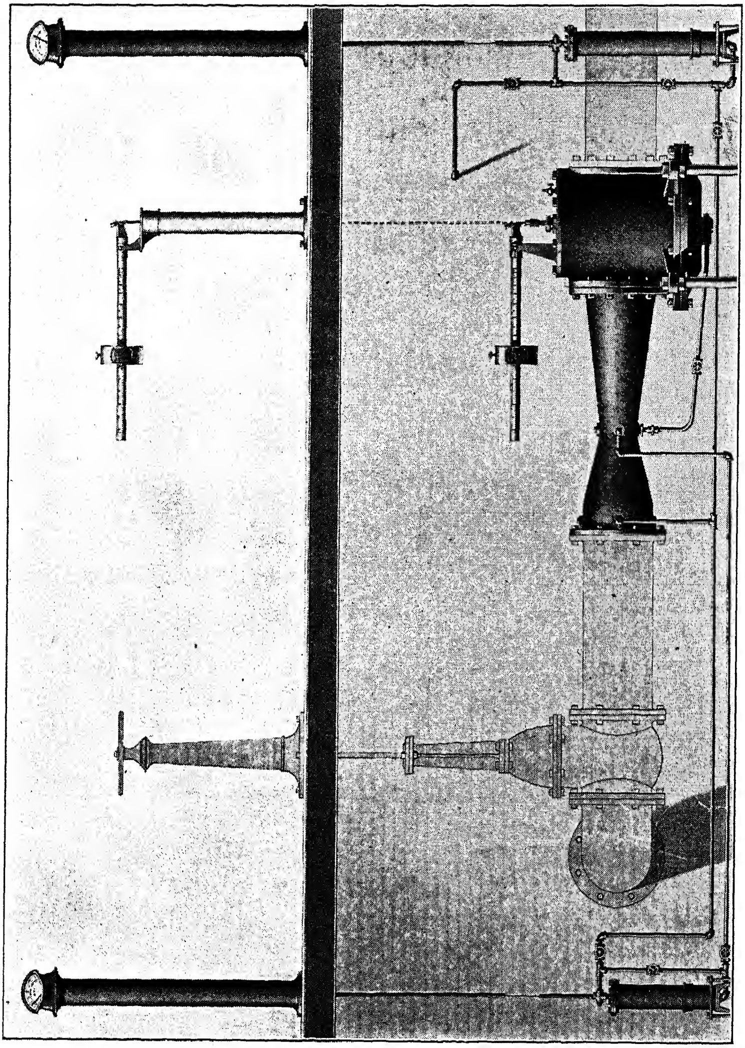

FIG. 81.—Arrangement of Simplex filter rate controller with Simplex rate of flow and loss of head gauges.

sure or pressure difference which varies with the volume of flow to be governed. Where a Pitot tube and piezometer are used the former carries water at a level corresponding to the static and velocity heads, while the latter carries water at a level equal to

the static head only. The difference in heads, therefore, represents the velocity head in the restricted throat and varies with the quantity of water passing through it.

The mechanism by which this principle is made to control the flow of water consists of three tubes in which hang cylindrical

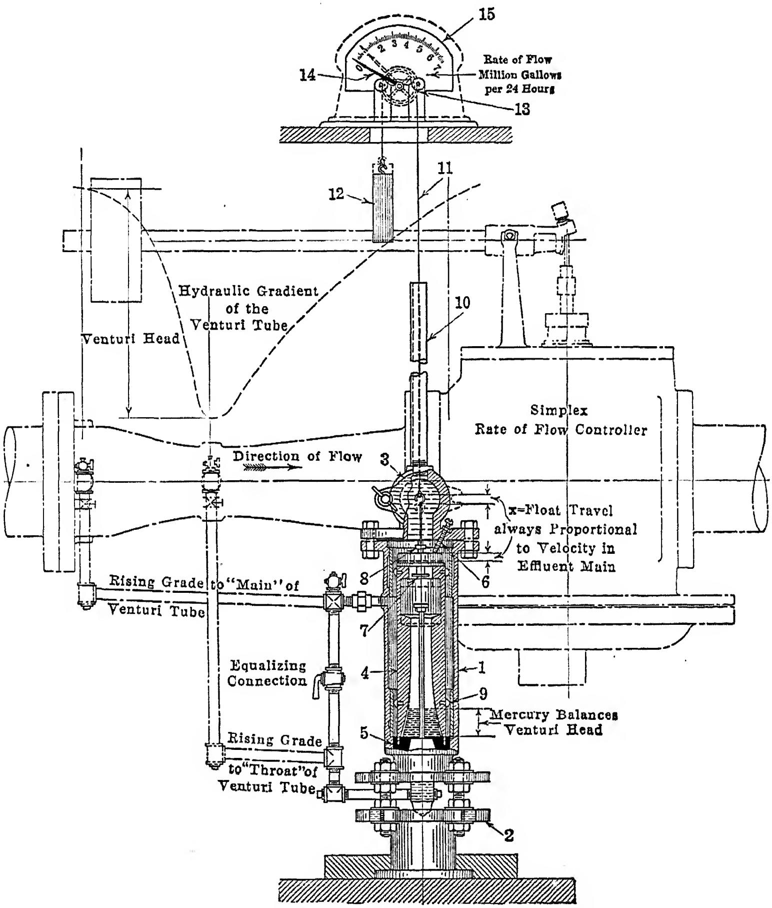

FIG. 81*a*.—Simplex rate of flow gage, showing connections to controller.

weights. These weights are suspended by rods from a bar supported on a knife edge. One end of the bar projects between two electrical contacts. When the bar is in contact with either terminal an electrical circuit is formed, whereby an electrically operated pilot valve causes the hydraulic throttling valve in the

effluent pipe to be either slightly opened or closed, depending upon the terminal in circuit. The cylindrical weights are in balance when the water about weights 1 and 2 (see diagram) stands at the same level, whether that level be high or low, and

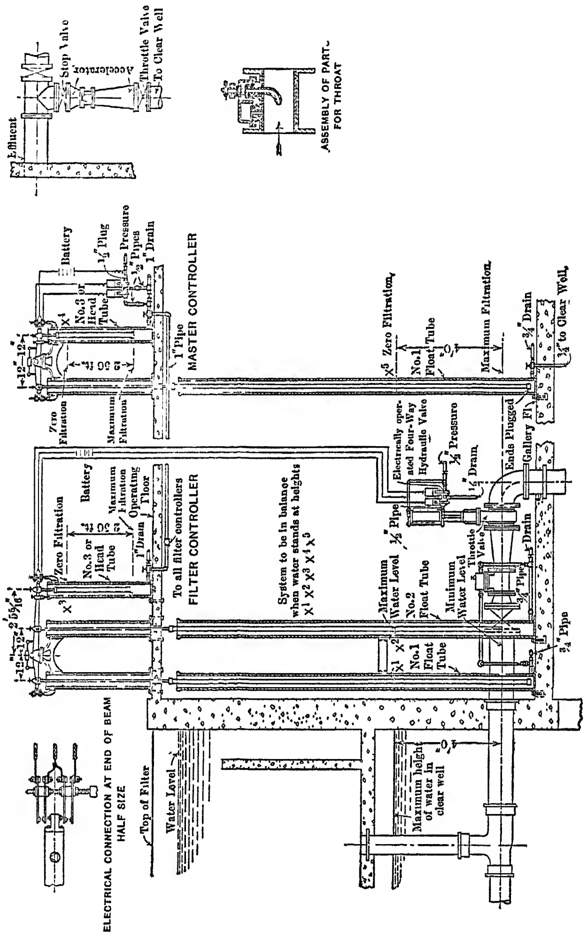

Fig. 82.—Earl rate controller.

the water about weight 3 stands at the level marked "zero rate of filtration."

By reference to the drawing (Fig. 82) it may be seen that tubes No. 1 and No. 2 extend above the maximum water level in the filters, and that they are connected to the Pitot tube and the piezometer, respectively, by pipes. Under operating conditions

the water assumes levels in these two tubes corresponding to the respective pressures in the Pitot tube and the piezometer. The

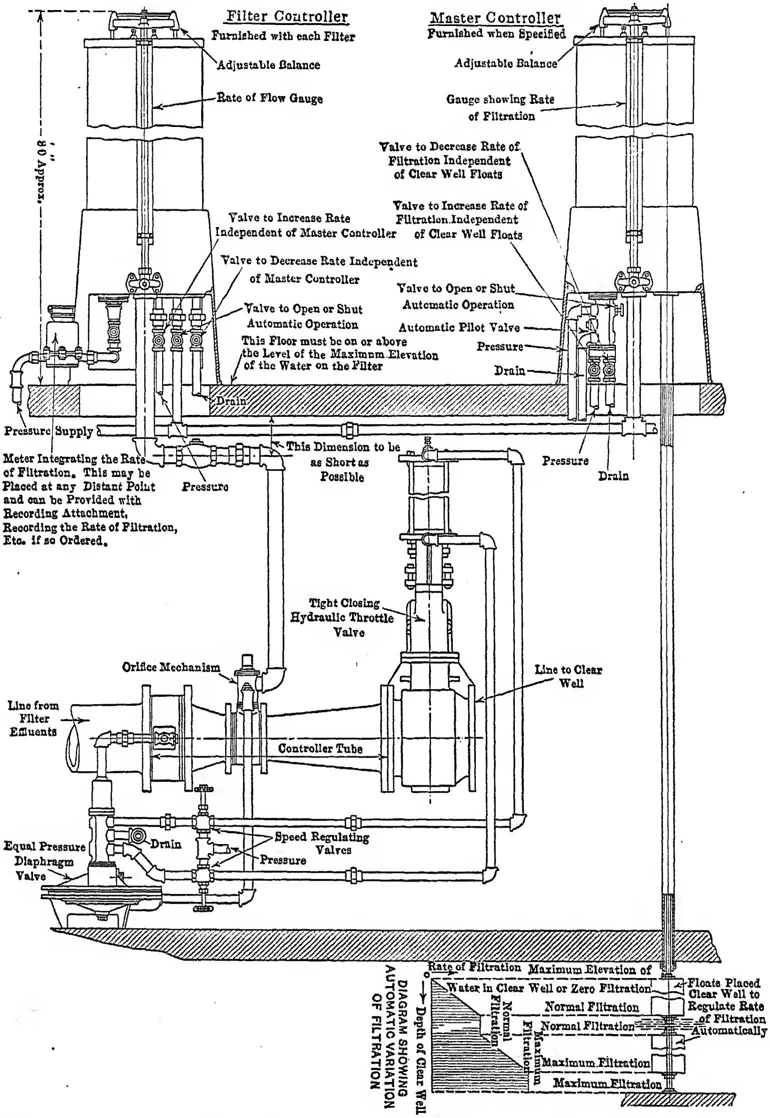

FIG. 82a.—Improved form of Earl rate controller.

weights in tubes 1 and 2 are of equal area, but both are weighted in excess of their maximum displacement. They are connected

through rods with the bar, which is supported on a knife edge, located on a stand placed on the operating floor level of the filters. The third tube with a cylindrical weight having about one-half the area of the other weights stands also on the operating floor. The suspended weights are adjusted in weight so that the beam is in balance when the water level is the same about weights 1 and 2, and at any assumed "zero rate level" about weight 3. With water in tube No. 3 at the "zero rate level," the controller would operate so as to entirely close the main valve. By holding any fixed level of water in tube No. 3 below the "zero rate level," a constant rate of flow may be maintained, for the reason that the set of weights will be in balance only when the water about weight 2 stands above the water level about weight 1 by a distance proportional to the aforesaid reduction in level about weight 3.

Earl Master Controller.—The Earl mechanism may be adapted to producing a variable rate of flow between given limits, by means of a "master controller." For this purpose a single tube connected to the filtered-water reservoir is placed at such a level as will permit the water to rise and fall within the limits desired; *i.e.*, between a minimum level below which it should not drop and at which point the filtration rate should be increased, and the maximum level in the reservoir above which the water must not go, and where filtration should cease. The weight hanging in this tube is connected by a rod to a bar supported on a knife edge, and is in equilibrium with another weight in a second tube standing at the operating floor level. By means of contacts above and below the projecting end of the balanced bar, circuits may be closed when the bar is out of balance, which open pilot valves admitting water to or from the upper tube, and thus restoring the balance. This tube is also connected with all the No. 3 tubes of the separate controllers, and thus regulates each individual filter, causing an increase or decrease in the rate of filtration according to the fall or rise of the water in the upper tube of the master controller, and corresponding to like fluctuations in the water levels in the lower tube and the filtered-water reservoir.

An improved form of Earl rate controller has recently been designed in which an equal pressure diaphragm is used in place of the long float tubes first employed. Through this diaphragm the pilot valves regulate the throttle valve of the controller. In other respects the principles upon which the control of the rate of flow is based are the same as in the original design. An

integrating rate of flow meter has also been made a part of this rate controller installation.

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CHAPTER XX

REGULATING, MEASURING AND INDICATING DEVICES FOR RAPID SAND FILTER PLANTS (CONTINUED)

MEASURING APPARATUS

It is important to know the rate at which water is passing through a rapid sand filter plant for several reasons. The operator should know the volume of water which is to be treated with chemicals in order that they may be applied in correct proportions. Knowledge of the velocity of the water flowing through the settling basins, and through the filters is necessary if proper periods of subsidence and rates of filtration are to be maintained. Legitimate uses of water about the plant, and losses by leakage are more readily controlled if measuring apparatus can be easily and quickly consulted. Such apparatus also serves as a check on pumping machinery which may be supplying to or taking water from the filtration plant.

Modern and well-equipped plants are, therefore, usually provided with a meter measuring the inflow to the plant, with rate-of-flow indicating devices on the filters, with a meter on the wash-water pipe line, and possibly with a meter on the main filtered-water effluent pipe line. Probably no one plant is equipped with all of these devices, since local conditions may make one or more of them unnecessary and still enable the operator to accurately control the operation of the plant. The most necessary meter is doubtless the one measuring the inflow to the plant. The measurement of the effluent is not so important, although it affords an excellent check upon the water used and lost by leakage in the plant. Measurement of the wash water is quite desirable, and may be determined in a number of ways, depending upon local conditions. For example, if water for washing the filters is drawn from a tank, it may be metered directly, or the volume used ascertained after each wash by the lowering of the water in the tank. If pumps are used to pump directly into the wash-water tank, the volume which they pump may be determined from the known capacity of the pumps. The employment of

rate-of-flow indicating apparatus on the filters is a refinement of control that can be dispensed with if reliable rate controllers have been installed. On the other hand, they may in a measure replace the use of automatic rate controllers, since hand regulation of the filters then becomes comparatively easy, although dependent upon the faithfulness of the attendant.

Venturi Meters.—The measurement of large volumes of water, such as must be handled in a filtration plant, offers more or less difficulty. Under exceptional circumstances weir measurements may be possible, but the more usual condition requires the measurement of the flow in pipe lines. For this purpose no better

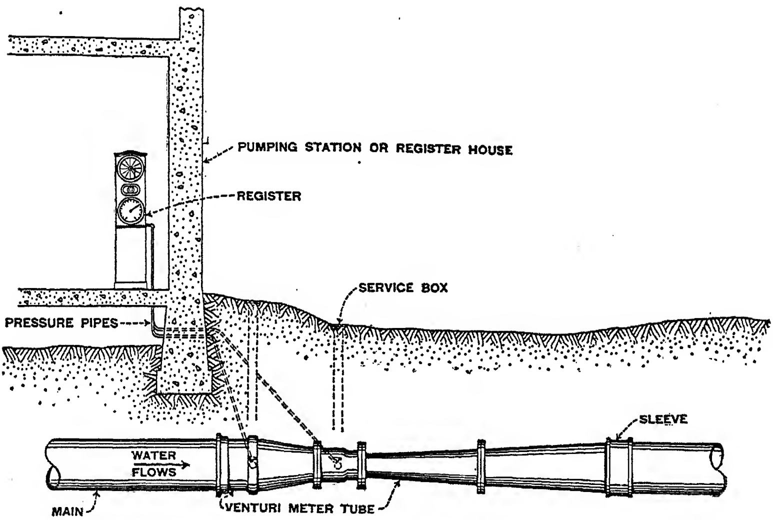

FIG. 83.—Venturi meter tube.

meter is at present available than a Venturi tube (Fig. 83). The well-known properties shown by water flowing through a pipe having a contracted section or throat joined to a converging cone on its upstream end, and a diverging cone on its downstream side, enable a fairly accurate measurement to be made of very large volumes of water.

As the water passes through the throat of the Venturi meter its velocity is increased, and its pressure decreased. A differential gage, connected on one side with the water at the entrance to the meter, and on the other side with the water in the throat of the meter, will indicate the loss of head due to the increased

velocity in the throat. There are a number of ways for making use of the general principles involved in order to obtain the rate of flow and the total volume of water which may have passed through the meter in any given period. Recording devices are also employed for making permanent records.

Where the registering apparatus must stand above a point to which the water in the pipe would rise, it is necessary to use floats which rise and fall in pipes connected to the upstream chamber of the meter and to the throat. Cords from the floats are wound around spindles attached to a differential mechanism located in the registering case. The floats are counterbalanced by weights. The same amount of rise or fall of water at the same time in both float tubes will have no effect on the movable hand registering the rate of flow on the dial face. With the floats at the same level the hand will point to zero on the gage. If the pressures transmitted cause the floats to stand at different levels, the hand will move to a point indicating the rate of flow.

Where the pressure of the water can be made to actuate the registering mechanism (Figs. 84, 84*a* and 85) without the intervention of floats, as above described, it is usually transmitted through mercury wells, which are subjected respectively to the pressures of the upstream chamber of the Venturi and to the throat. In each mercury well is a heavy metal float which rests upon the mercury. The mercury flows from one well to the other in direct proportion to the difference in the two pressures; consequently one float rises as the other descends, and this movement is transferred through a rack and spur gearing to the indicator dial hand-shaft. A cam on this shaft controls the position of the pen on the recording chart, and also the amount of movement of the counter-dial figures, where a register which totals the quantity of water passing through the meter forms a part of the instrument.

Where Venturi meters can be installed under the most favorable conditions, they are accurate to within 1 or 2 per cent. If, however, local conditions are unfavorable, or if they do not receive the moderate amount of care that is required to keep them in good working order, they may vary as much as 5 or 6 per cent. from the true rate of flow. Where muddy waters are being measured, the inlet pipes and annular throat chamber should be blown out quite frequently in order to prevent the vent holes to the throat becoming clogged. The tubes in which the floats rise and fall should be blown out occasionally as well as all connecting pressure

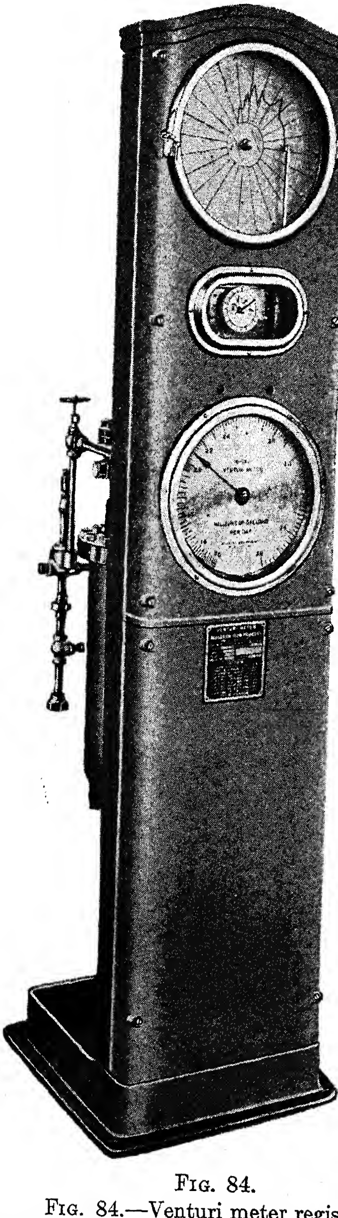

FIG. 84.

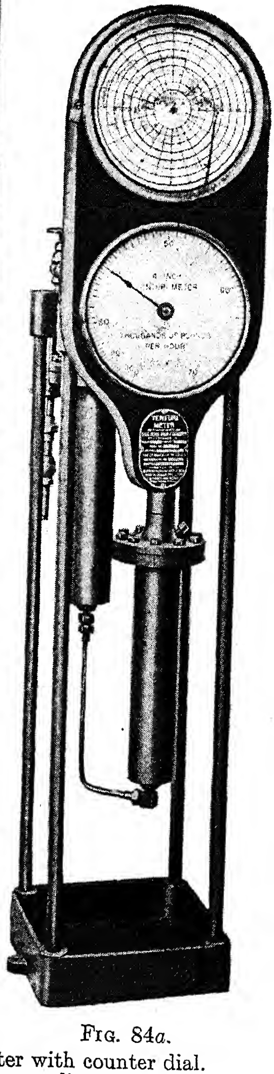

FIG. 84a.

FIG. 84.—Venturi meter register with counter dial.
FIG. 84a.—Venturi meter register with recording and indicating dials.

pipes. The differential mechanism requires a slight amount of oiling from time to time; and when cords attached to the floats break a readjustment of the spindles and cam is usually necessary. The clockwork which carries the recording charts needs some attention in the way of oil and regulation, and the pens should be kept clean and full of ink. Air may accumulate in the water pressure piping where mercury wells are used, and should be released when this occurs through the vent cocks provided for this purpose.

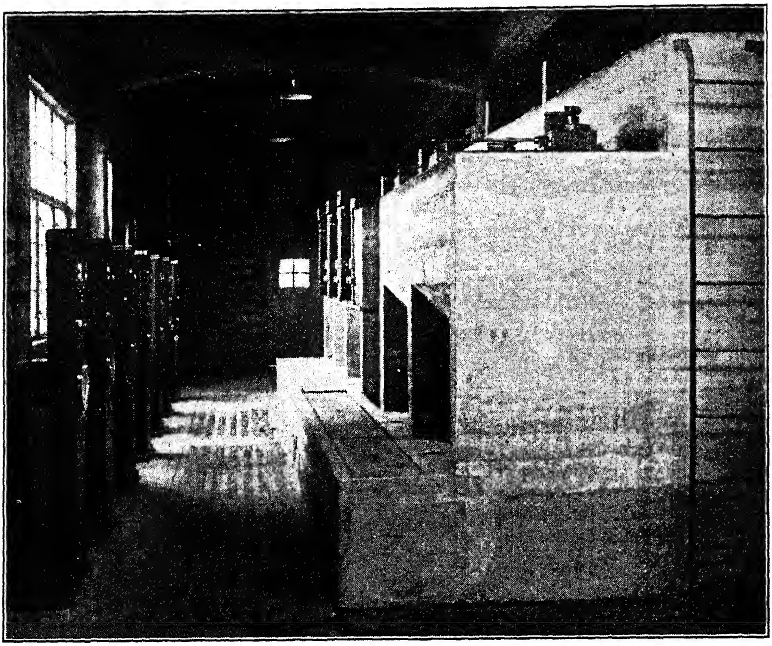

FIG. 85.—Venturi meter registers for St. Louis filter plant chemical feed controllers.

A Venturi meter manometer is a simple and comparatively inexpensive instrument for measuring the rate of flow in Venturi tubes. It is essentially a U-tube partly filled with mercury, one side of the tube being connected with the inlet and the other with the throat of the Venturi tube. The rate of flow may be read directly from a graduated scale placed between the two mercury columns. The zero of the scale is placed on a level with the lower mercury level, and the rate is read opposite the upper mercury level. These scales are, of course, graduated for the

particular Venturi tube through which the rate of flow is being measured. The instrument is not suitable for measuring rapid fluctuations in the rate of flow.

Simplex Water Meter.—In this apparatus especially adapted registering instruments make use of the principles of a Venturi tube or of a Pitot tube in order to measure the flow of water in a pipe. It may also be adapted to a weir, canal or conduit if suitable pipe connections can be made between these open channels and the registering apparatus.

The registering apparatus (Figs. 86 and 87) consists of a mercury float chamber, containing a float of such variable cross-

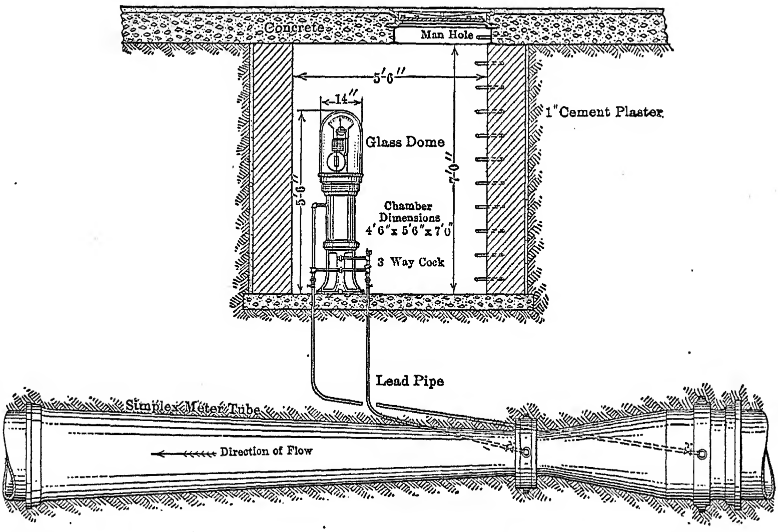

FIG. 86.—Diagram of a Simplex meter register connected to a Venturi tube.

section that its movement is in direct ratio to the flow of water through the Venturi tube, pipe, conduit or weir. The movement of the float actuates a revolving shaft to which is attached a hand which moves over the face of a fixed dial having uniform graduations. Attached to the shaft and moving in proportion to the angular deflections, is a pen in contact with a rectangular chart wrapped on a revolving cylinder. A traction wheel passes over the face of a revolving disk, and is geared to a train of wheels operating a series of small dials, similar to those of an ordinary gas or water meter. The cylinder and disk are operated by a clock. The traction wheel is so constructed that its movement,

while in contact with the face of the disk, is free from rubbing friction. As the movement of the traction wheel and the pen are in direct ratio to the angular deflection of the shaft, they are also in direct ratio to the movement of the float and of the flow of water.

The chart is uniformly graduated, the abscissas representing hours and the ordinates the rates of flow. The area comprised by the datum line and the line described by the pen on the chart,

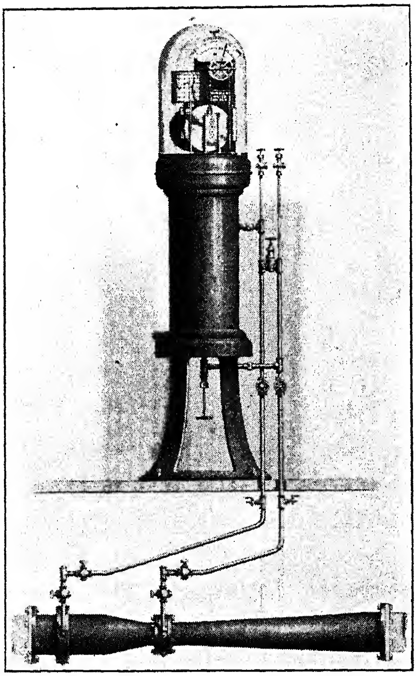

FIG. 87.—Simplex meter register and recorder, showing connections to Venturi tube.

is in direct ratio to the total flow, and by determining this area and multiplying by the coefficient given on the face of the chart, the total flow may be calculated.

Rate-of-flow Gages.—Gages indicating the rate of flow of water through a filter are useful where close regulation is desirable. Apparatus of this type may be readily installed where a Venturi-tube type of rate controller is used, since the Venturi affords a means of indicating the rate of discharge when used in connection with appropriate registering apparatus. By the use

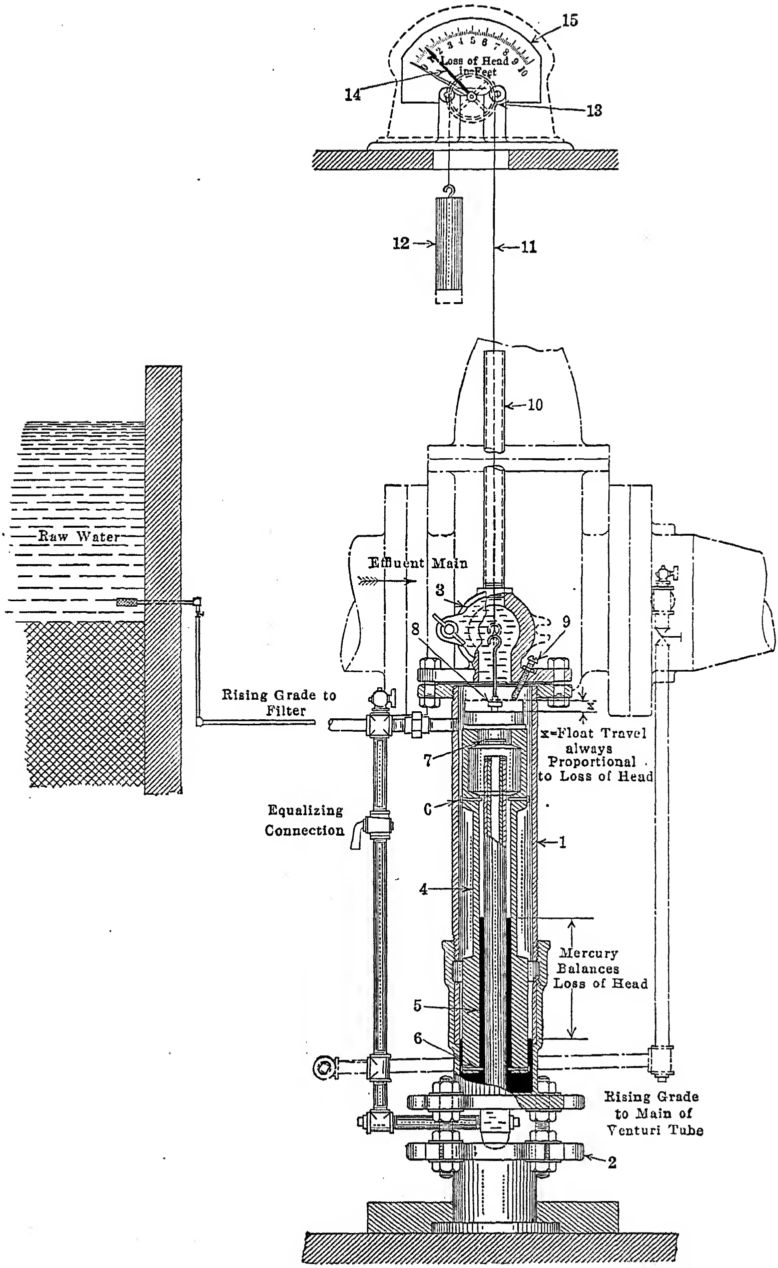

FIG. 87*a*.—Simplex loss of head gage, showing connections to filter and effluent piping.

of a Pitot tube it is also possible to determine the rate of flow of water through a pipe, where no Venturi tube forms a part of the effluent piping.

Loss-of-head Gages.—A knowledge of the frictional resistance offered to the flow of water through a filter by the accumulating sediment being strained out of the water is valuable to the operator of a filter. By noting the loss of head when the filter is started, and also when it is no longer able to maintain the rate of flow at which it is intended to be operated, the head lost becomes a measure of the useful period of service of the filter. Where filters are operated at variable rates, this information is of somewhat less value, unless recording instruments are employed, whereby the relation between the losses of head during successive portions of the period of service may be compared.

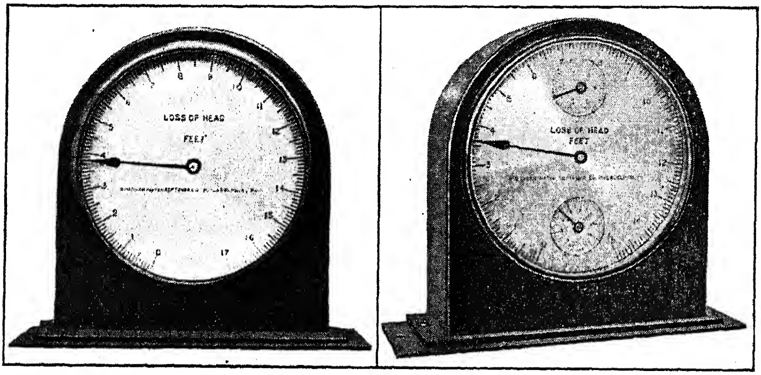

FIG. 88.—Loss of head gages.

The loss of head is measured by a gage, which records the difference in level of the water above the sand line in the filter, and that to which it will rise in an open pipe connected directly to the effluent pipe of the filter. This is commonly effected by the use of floats placed in vertical tubes connected directly to the portions of the filter mentioned above. Cords from the floats pass over spindles connected to a train of gears, which operate hands rotating in front of dials marked with the two water levels in feet. In some gages a separate indicator (Fig. 88) is used for each water level, while a third hand indicates the difference between the two levels. The floats are counterweighted. Silk cords or a small flexible copper-wound cable is used to transmit the movement of the floats to the train of gears. The gage is

usually placed on the operating floor level, and at some convenient point so that it may be easily observed.

In one type of loss-of-head gage a hollow float is inverted in mercury contained in the bottom of a cylinder. The float can move vertically and transmits its motion through a rod passing through the top of the cylinder. The space in the cylinder outside the float is connected by a pipe with the water standing above the sand level in the filter, and the inside of the float above the mercury is connected by a pipe with the effluent main. Differences in pressure are transmitted through the mercury, and cause the float to rise and fall. The movement of the float is transferred through the rod mentioned above to suitable registering apparatus.

A type of loss-of-head gage which uses no floats, but draws the water to a mercury manometer by means of a vacuum pump, is also used. One leg of the manometer is connected through piping with the water on the surface of the filter, and the other leg with the water in the effluent pipe. The decreasing pressure of the water in the effluent main, from the time the filter is started to the close of its period of service, is measured directly on a gage by the rise of the mercury column connected with the effluent pipe.

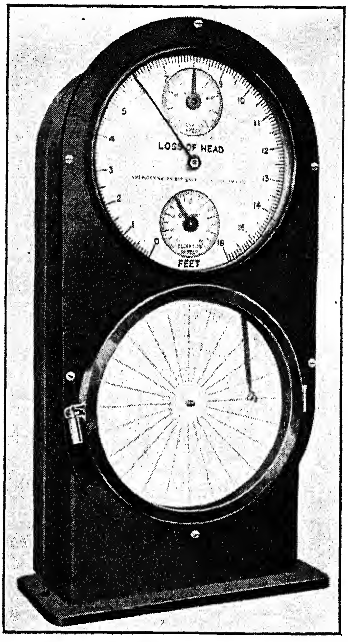

Fig. 89.—Recording type of loss of head gage.

Recording types (Fig. 89) of loss-of-head gages are useful if a careful study of the operation of the filters is desirable. They are by no means as necessary a part of the equipment as are the non-recording types. If sufficient attention is not given to them, so that they are kept in good working order, they are, of course, useless.

Water-level Gages.—It is usually necessary to know the water levels in reservoirs, basins and tanks for the proper control of the operation of purification works. Where possible, ordinary pres-

sure gages are usually sufficient, if properly placed. Recording gages are frequently desirable, as they give a continuous record and enable the operator to keep a permanent record of fluctuations in the water levels.

Gages should be grouped together as much as possible, since water levels in different parts of the system may be thereby more conveniently compared. Where the gage is placed at a point below the water level it records, ordinary spring or diaphragm gages may be used. These gages had best be graduated in feet above a known zero elevation rather than in pounds, since comparisons of water levels in different parts of the system are thereby more easily effected.

When the gage must be placed above the water level to be recorded, electrical gages may be employed. An excellent type of gage of this character consists of a float which rises and falls in a tube connected to the water level to be measured. A cord attached to the float passes over a small wheel connected with a transmitting device. The movement of this wheel operates the mechanism inside the transmitter, and causes a momentary closing of one set of contacts for a certain predetermined movement of the float upward, and a similar closing of another set of contacts when the float drops. The transmitter can be made to close the contact ten or any other convenient number of times for a change in elevation of 1 ft. by the float.

The transmitter is connected to the receiver by three wires. The middle wire acts as a common return for the current to both sets of contacts. The two outside wires are connected to two separate sets of electromagnets in the receiver. When the armature of one set is drawn toward its magnet by the closing of one set of contacts in the transmitter, it moves the pointer and recording pen in one direction. The other set of magnets operate the pointer and recording pen in the opposite direction when the circuit is closed through the second set of contacts in the transmitter.

The batteries used to operate the electromagnets may be placed either at the receiver, or at the transmitter, or at any convenient place on the circuit between the two.

The above-described gage, which is known as the "George E. Winslow Electrical Indicating and Recording Apparatus," is made in different styles to meet varying conditions. In some cases a sensitive pressure diaphragm, which operates the trans-

mitter by means of a lever, may take the place of the float and counterweight.

A similar long distance, electrical, indicating and recording apparatus (Fig. 90) is made by the Bristol Co. although based

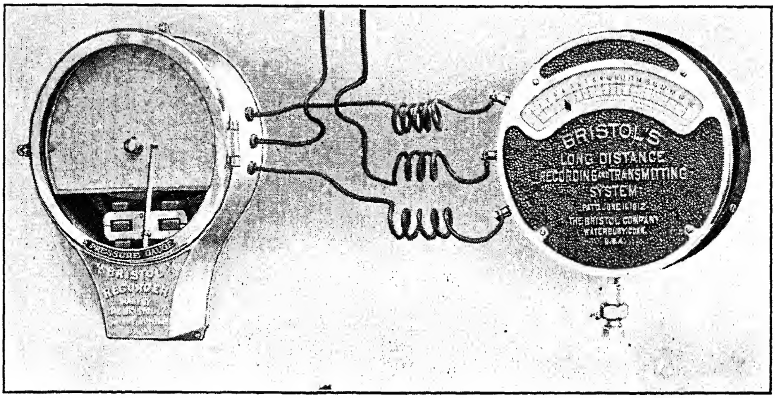

FIG. 90.—Bristol recording water-level gage.

upon quite different principles. The principle used in this system of transmission is that of an induction balance. The apparatus consists of a sensitive diaphragm bulb, which is submerged

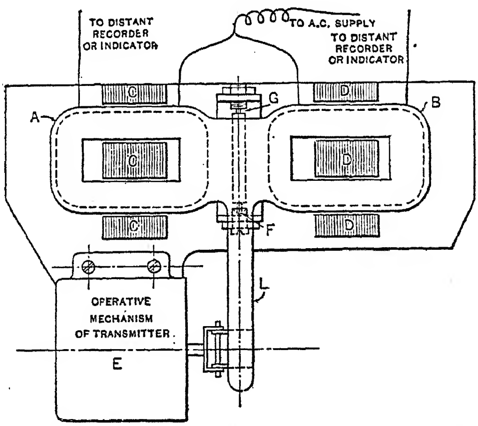

FIG. 90*a*.—Bristol recording water-level gage showing electrical connections.

in the water to some known level; a copper tube connection between the bulb and a transmitter; and a three-wire circuit connecting the transmitter with the receiver and indicating and

recording gages. It is necessary to use an alternating current to operate the transmitter and receiver.

The transmitting and receiving instruments are both equipped with two pairs of coils, arranged to swing in a horizontal plane over laminated iron cores. In the transmitter the pressure transmitted by the sensitive bulb through the copper tube, moves a helical pressure tube, which in turn causes the mechanically balanced solenoids to be deflected. One solenoid moves off from its core as the other moves on over its core, thus increasing the inductance in one solenoid and decreasing it in the other, and thereby proportionately increasing the flow of current in one coil and decreasing it in the other.

The variations of current at the transmitter will produce like variations at the receiver, and cause the pair of solenoids of the

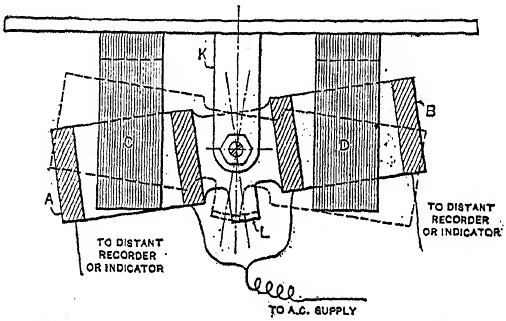

Fig. 90*b*.—Bristol recording water-level gage showing balanced solenoids.

receiver to seek positions corresponding to those of the transmitter. The movement of the receiving coils is transmitted through an appropriate mechanism to a pointer moving over a properly calibrated scale, or to a pen which can trace a line on a chart for the purpose of producing a permanent record.

Water-sampling and Inspecting Devices.—Means for obtaining samples of water from each individual filter are usually desirable in the operation of rapid sand filter plants, since the effluent of one filter, if it is operating improperly, may contaminate the output of a whole group of filters. For example, where muddy waters are being filtered, the turbid effluent of one filter may cause a slight opalescence in the combined discharge from all the filters, in which case it becomes necessary to detect which filter is producing the poor effluent in order that it may be taken

out of service. By sampling the water from each filter the one producing the poor effluent may be easily discovered.

Sampling devices for each filter commonly form, therefore, a part of each filter's equipment. They consist usually of a small pump with its suction pipe connected to the effluent pipe of the filter. The pump is usually made to discharge the water through a faucet into a sink located in the filter operating table. The pumps are either electrically or hydraulically operated, and are started and stopped from the operating table.

An elaborate sampling apparatus, consisting of a multiple pump centrally located with respect to the filters, and having as many pump cylinders as there are filters, so that water may be drawn from any filter desired, has been used in some plants.

Sight tubes of glass through which water flows continuously from both the influent and effluent mains afford a method of observing the character of the water applied to the filters as well as that being discharged from them. They form a rapid and easy means for gaging the operation of the plant, if located in some portion of the filter-plant building where they may be frequently and readily observed.

The multiplying of devices for sampling or observing the character of the water passing through a plant can be easily overdone. On the other hand, wherever the character of the water being pumped is undergoing rapid variations, which demand frequent changes in the quantity of the chemicals required to properly purify it, as well as modifications in the general method of handling it, the operator should be provided with all the necessary registering and sampling apparatus that are needed. False economy in this respect is as bad as extravagance in the opposite course.

Where economy in the installation of a plant is necessary, an ordinary faucet tapped into the influent and effluent pipe line of each filter tank is usually sufficient to supply the samples which may be needed from each filter. Some inconvenience in getting to such sampling places, because of their location in the pipe gallery, is more than offset by the relatively infrequent intervals at which such sampling is generally necessary. In practice, samples of the water being applied to the filters, and of the combined effluents from the filters, are usually quite sufficient for routine control, and for this service sample pumps should be installed for the collection of water at these points.

Operating Tables.—The operations of starting, stopping and washing a rapid sand filter are most easily carried out at some point where the movement of the water in the tank can be observed. As this point must be at the top of the tank, and as the valves which control the above operations are at a somewhat lower elevation, it necessitates apparatus adapted to these conditions. Where hand-operated valves are used the extension

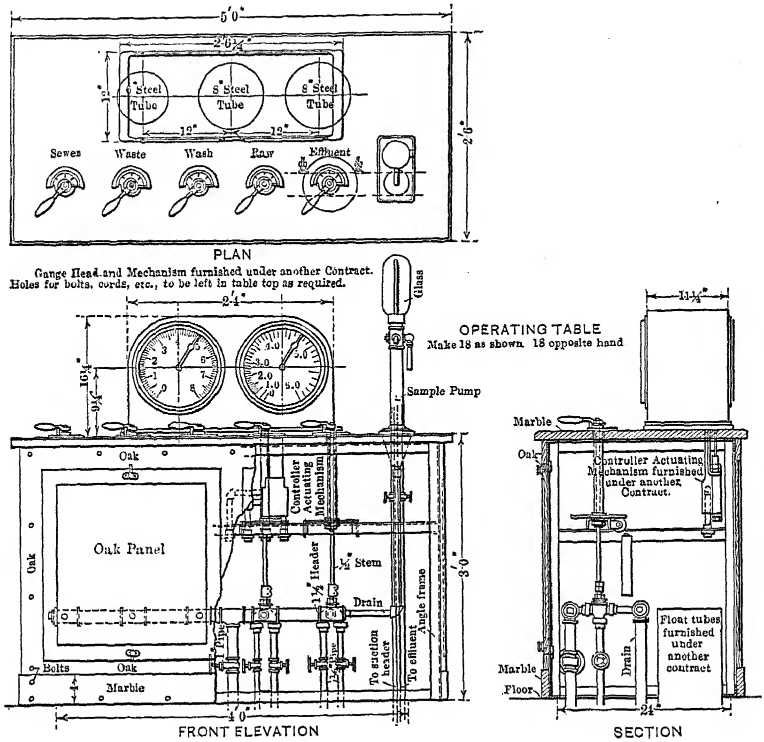

FIG. 91.—Plan and elevation of filter operating table.

of the stems to the operating floor level does not always lend itself to a convenient grouping of the wheel stands, because of the positions which the valves must occupy on the pipe lines below. However, in small tank units the stands are quite close together, although somewhat irregularly spaced, and produce a convenient and inexpensive design. Separate stands for loss-of-head and rate-of-flow gages may also be provided in this arrangement without the use of tables.

In the more modern plants, where hydraulically operated valves (Figs. 91, 92 and 93) have taken the place of hand-operated valves, it has become the custom to place the controlling apparatus on a small table. This table or cabinet is provided on its top with a series of levers and indicators, which by means of multiple-way cocks located within the cabinet, move and indicate the position of the hydraulic valves. The piping from the hydraulic valves is brought up to the table and connected to the controlling cocks. Cords or wires passing over pulleys connect the tail-rod of the hydraulic valve with the indicator

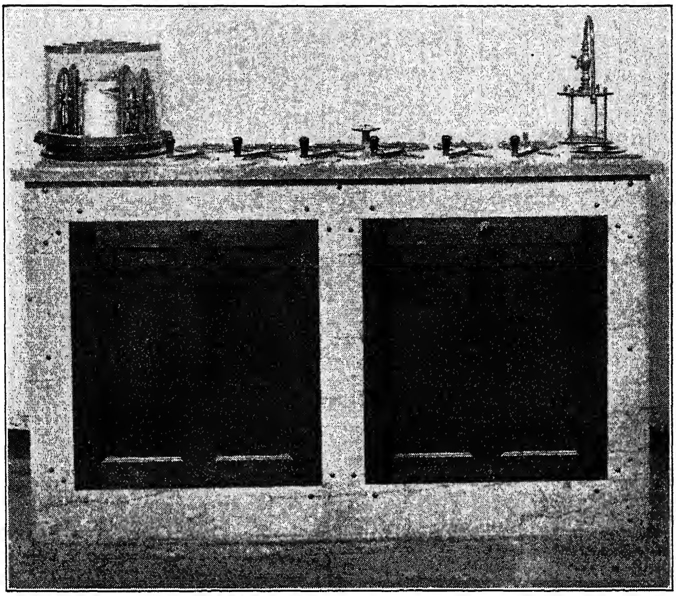

Fig. 92.—Filter operating table used in Minneapolis, Minn., filter plant.

on the table. The indicator can thus be set to show the exact position of the valve.

The loss-of-head and rate-of-flow gages, when the latter gage is used, are also usually located on the top of the operating table, or on a board fastened to the back of the table. Sampling faucets to which water is forced by small pumps, and in some cases sight tubes by which the character of the influent and effluent water may be observed, sometimes form a part of the apparatus on an operating table. In such cases bowls or sinks must be provided for waste water.

In the case of electrically operated valves the assembling of the switchboard panels, from which the valves are moved, side by side at some convenient point at the top of the filter, takes the place in a measure of an operating table. A stand or table is in such cases required for holding the gages and sampling apparatus.

Operating tables are usually constructed with marble tops and with hardwood paneled sides and front. The metal work of the controlling and indicating apparatus is usually nickel-plated, and the piping and cocks are generally of brass.

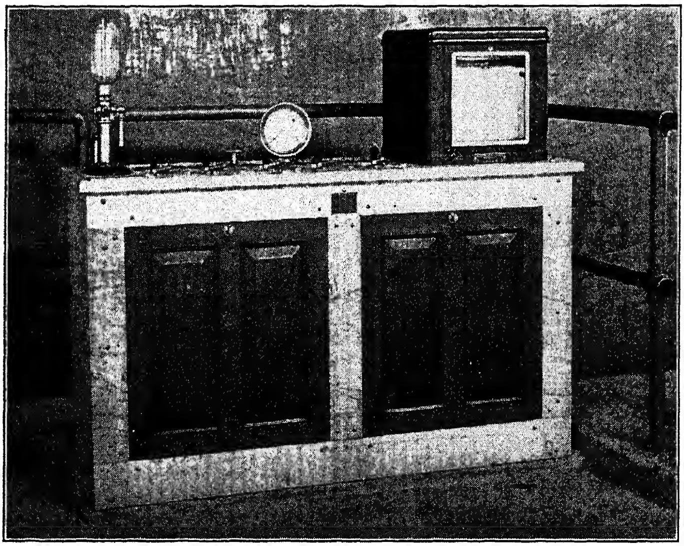

Fig. 93.—Filter operating table used in St. Louis, Mo., filter plant.

The centralizing of the control of the operation of a series of filters or of all the filters in a plant at one point has been tried but with no very great success. The principal reason for the failure of this method is that the operator can not see the water in the tank which he is operating.

It is evident that the use of all the operating-table equipment described above, makes the handling of the filters easier; nevertheless, its installation without due regard to the real needs of the plant may lead to extravagance of design not commensurate with the results which could usually be expected.

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CHAPTER XXI

EQUIPMENT FOR THE HANDLING AND STORING OF CHEMICALS AND FOR THE PREPARATION OF SOLUTIONS

The handling and storage of chemicals used in the opreation of water-purification plants call for careful consideration on the part of the designer. The uninterrupted operation of the plant is usually necessary, and, in consequence, a constant supply of chemicals must be provided. Plants are not infrequently so located that direct railroad connections are impracticable, in which case dependence must be placed on hauling the chemicals by wagon with horses or with motor-driven trucks from the railroad to the plant. This latter method is entirely practicable for keeping small plants supplied, but will hardly answer for the large plants, where the quantities of chemicals used daily are measured in tons rather than in hundredweights. At the large plants railroad connections are virtually imperative, unless the plant happens to be so situated as to be able to receive its supplies by water.

Whether railroad facilities are available or not, there should be ample storage capacity at the plant to care for its operation for at least 4 to 6 weeks, if, for any reason, its regular supply of chemicals should be cut off.

The manufacture at the purification plant of certain of the chemicals used in water-purification processes has been proposed. In several plants plans for making sulphate of alumina from raw material have been completed, and the necessary apparatus has been purchased and installed. Only under rather exceptional conditions could such a practice be possible, and then only in the large plants where competent and experienced chemists could be placed in charge of the manufacturing.

Chemicals Used and Their Transportation.—The chemical compounds commonly used in water purification are sulphate of alumina, sulphate of iron, caustic and hydrated lime, and carbonate of soda or soda ash. For disinfection purposes, calcium hypochlorite or bleaching powder, and liquefied chlorine

gas are required. Copper sulphate in small quantities is also sometimes used for destroying minute microscopic plant growths.

Some of these chemicals, like caustic lime, are best handled in bulk, although it is not uncommonly shipped in barrels. It has been handled in sacks of cotton duck with a fair measure of success, if the lime is first crushed before being placed in the bags, and if the period of transportation is not too long. Hydrated lime can be obtained in paper sacks. Lime in this form packs closely and does not carbonate readily. On the other hand, its cost is greater than the caustic lime. As it contains about 30 per cent. of water, the freight charges for this amount of water add materially to the first cost.[1]

Sulphate of iron is handled in bulk and in sacks, and the same is true of soda ash. Sulphate of alumina may be handled in bulk or in sacks, but is more frequently transported in wooden barrels. Copper sulphate is shipped in barrels.

Bleaching powder is usually obtained in sheet-iron drums containing 500 to 600 lb. of the chemical. This chemical is also packed in wooden casks. The latter weigh about 300 lb. Sheet-iron drums holding about 100 lb. may also be obtained, but the larger drums containing five or six times as much are more common.

Liquefied chlorine gas is transported in steel drums each holding about 100 lb. of the gas, or in larger cylinders which hold as much as 2,000 lb., and which weigh with the steel drum over 3,000 lb.

Best Methods of Handling Chemicals.—The best method of handling the various chemicals employed in water purification depends largely upon the properties of the chemical, the quantities to be used, and the storage facilities. Chemical compounds that are hygroscopic must, of course, be protected as much as possible from moisture. Handling large quantities of chemicals like sulphate of alumina, sulphate of iron, or lime in bulk is, obviously, the cheapest method to follow, although it is not always practicable to do so. The smaller the package, as for example where the material is barrelled or bagged, the higher the cost and the easier it is handled. Convenient sizes for packages range from 100 to 400 lb. in weight, but if above 400 lb. the package becomes awkward to handle. For example, the customary

[1] Lime carbonates only after it has been hydrated; hence when kept in air-tight bins it does not deteriorate.

weight of bleaching-powder drums is from 500 to 600 lb., which makes them somewhat troublesome to handle. On the other hand, packages weighing as high as 400 lb. can be quite easily moved on a level by one man on two-wheeled barrel or bag trucks.

The elevation of chemicals from the unloading platform to storage space, near the top of the solution tanks, is usually necessary in all plants. Where the chemicals are received in bags or barrels, the ordinary endless-chain bag or barrel hoist operated by a small electric motor furnishes the easiest and quickest method of handling large quantities of chemicals. In small plants, a good-sized platform elevator operated by either hydraulic power or by an electric motor probably answers the requirements better than an endless-chain bag or barrel hoist.

Unloading, Elevating and Conveying Chemicals in Bulk.—In those plants where large quantities of coagulating or softening chemicals are used, the delivery is frequently made in bulk in box cars. Lime and sulphate of iron have been successfully handled in this manner. Where lime is bought in bulk it is usually necessary to crush it before discharging it into storage rooms or bins. In such cases the lime is shoveled by hand or by an automatic power shovel directly from the car into the crusher, from which the lime drops into an elevator. Bucket elevators, chain drag and screw conveyors, and belt conveyors have been used for hoisting and conveying the lime to the storage space. Belt conveyors have been tried for both lime and sulphate of iron, but have not proven satisfactory, since it was found that the dust which collects in the casings below the belt increased the power required for operation, and if not cleaned out at regular intervals eventually stopped the machinery.

In general it may be said that the simpler methods of handling chemicals are more reliable, although they involve more manual labor. This latter element of cost does not become of much consequence until the quantity of chemicals to be handled daily reaches so large an amount as to require a considerable force of men; in which case mechanical methods are probably cheaper.

In selecting machinery for handling chemicals for any plant, preference should be given to that which is composed of the fewest and simplest working parts. The breaking down of this class of machinery may cause serious inconvenience in operating a purification plant, and its upkeep may become burdensome when a too elaborate equipment has been installed.

The dust, which is practically unavoidable in handling nearly all of the chemicals employed in water purification, should be minimized as much as possible. Lime when ground or crushed, or when hydrated, is particularly disagreeable on account of the fine dust which gets into the air wherever it is being handled. Adequate ventilating systems should always be provided where much material is being moved in order to lessen as much as possible the volume of dust escaping to the air, and thereby reduce to a minimum the discomfort of the workmen who must handle it.

The Storage Plant for Chemicals of the St. Louis Water Department.—A very large storage plant for lime and sulphate of iron was constructed and placed in service in 1909 at the St. Louis water-purification plant.

Building.—The building is divided into three sections (Fig. 94), the storage bins comprising the two end sections, and the operating department the third or central section. The cylindrical bins for the lime are at one end of the building, and those for the iron sulphate at the opposite end. Four bins are provided for each chemical. The capacity of these bins is approximately 10,800 cu. ft., each, giving a total storage capacity of about 1,250 tons of lime, and 1,600 tons of sulphate of iron, equal to a supply for 37 days of the lime, and for 115 days of the sulphate of iron at the rate at which the chemicals were being used at that time.

The central or operating section of the building has two floors above the basement, one at the ground level called the pump-room floor, and a second or mixing-room floor. A lantern at the top of the building crosses the mixing room and partly covers the bins on either side. In the basement are the crushers for both lime and sulphate of iron, most of the conveyors for storage and reclaim, air compressor, pump, sump-pit, and a system of piping. On the pump-room floor are the heater tanks, pumps, switchboard, wash and locker rooms. On the mixing floor are the tanks for slaking the lime, and for making the sulphate of iron solution, and the auxiliary measuring and weighing devices.

Crushing and Conveying Systems.—There are two unloading points for lime, and one for sulphate of iron. At each of the unloading points for the lime is a No. 2 Sturtevant crusher with a capacity of about 15 tons per hour. Each crusher with its side conveyor is driven by a 30-hp. motor. The crusher for the sulphate of iron is a No. 1 Williams hammer crusher driven by a

15-hp. motor. The lime is shoveled into chutes which convey it to the crushers. From the crushers the lime is carried to elevators which discharge on to a scraper conveyor, from which

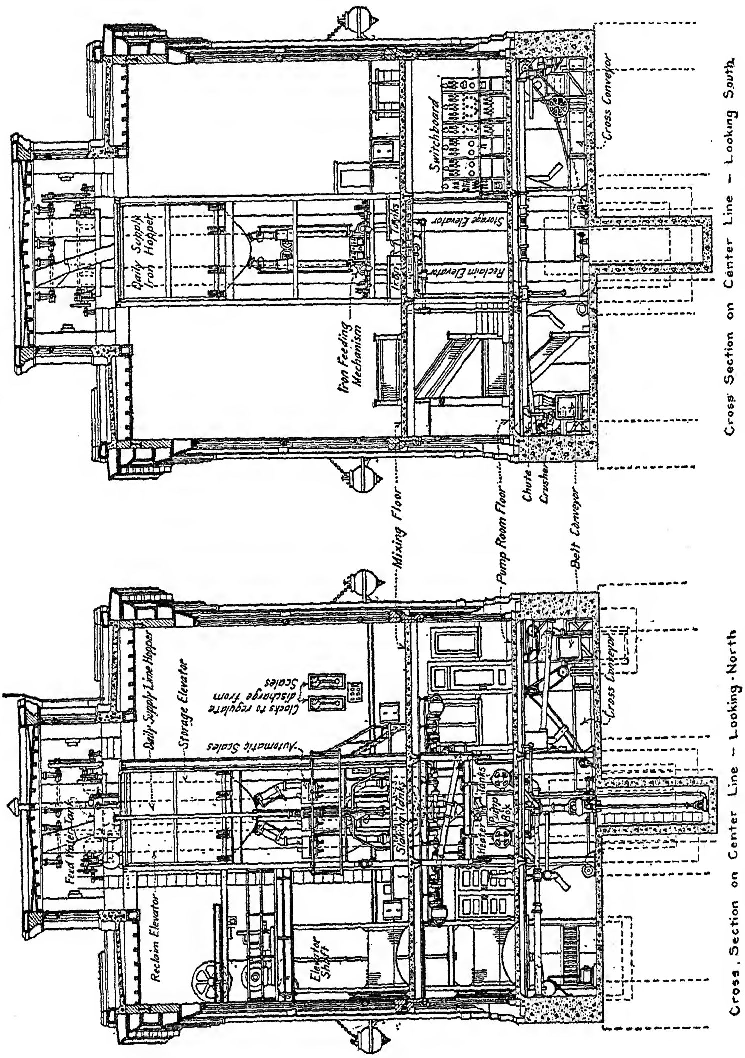

FIG. 94.—Cross-section of center of coagulant house in St. Louis filter plant.

the lime is distributed to the storage bins. The elevator and scraper conveyor in the lantern are driven by a 7½-hp. motor.

In reclaiming the lime from the storage bins, it is taken from

the bottom of the latter, and by means of conveyors carried to an elevator, which raises it again to the lantern, and discharges it directly into an auxiliary storage tank or daily supply hopper set on columns in the mixing room above the slaking tanks. This daily supply hopper holds from 25 to 30 tons of material.

Difficulties Encountered in Operating Equipment.—The belt conveyors first employed in the St. Louis plant were found unsuited to handling either the lime or iron sulphate on account of the collection of dust in the casings below the belt. They were replaced by conveyors of the scraper type.

Another difficulty which was encountered in the storage of the sulphate of iron in the bins was due to the iron sulphate crystals cementing together and thus causing the mass to cake. Bins from which material was irregularly drawn caked worse than in those which were being constantly used. Moisture left in the chemical on account of improper drying was thought at first to be the only cause of the trouble; but it seems probable that the cementation of the crystals is in part due to the escape of water of crystallization and to the pressure to which they are subjected in the storage bins on account of the great mass of material stored in them.

These cakes of sulphate of iron, if they were not broken up in the conveyors and elevators carrying the chemical to the daily supply hoppers, clogged the orifices of the feeders and necessitated constant attention. The installation of grinders just following the discharge of the chemical from the supply hoppers made the uniform feeding of the sulphate of iron through the orifices more certain, but did not remedy the trouble entirely even at this point.

In order to reduce the caking in the bins as much as possible and to avoid the disagreeable necessity of sending men into the bins to break up the caked mass, an experimental chain breaker was installed in one of the bins. It consisted of a horizontal girder supported from the bin roof, and geared to make two revolutions per minute, when driven by the reclaim conveyor motor, and carrying a heavy chain whose length was adjusted so that it scraped over the surface of the cemented iron sulphate. The chain loosened the iron sulphate so that disagreeable labor of breaking it up by hand in a suffocating dust was to some extent avoided. However, lumps too large to pass the discharge chute of the bin leading to the conveyor required them to be dislodged

and broken by poking the material through bar-holes in the conical-shaped bottom of the bin. The chain breaker proved of no value if the material had been stored too long in the bin.

Mr. W. F. Monfort in discussing the probable reasons for the caking of sulphate of iron crystals states that it is well known that at 70°F. crystals of iron sulphate of the composition $FeSO_4$-$7H_2O$, subjected to a stream of air, lose 3 molecules of water, and pass over into pale green crystals of the composition $FeSO_4$-$4H_2O$. By heating to higher temperatures oxidation of the salt commences and finally all of the water of crystallization is driven off.

The loss of the first 3 molecules of water takes place at summer temperatures to a limited extent during the period of shipment, so that the material has a greenish-gray appearance. Its composition is only slightly affected, but it is sufficient to cause the analyses to indicate a little more than 100 per cent. of the salt, $FeSO_47H_2O$. The partially dehydrated crystals, produced in the exposed layers in transit, become distributed throughout the mass when the shipment is transferred to the closed bins. If the temperature in the bins rises above 70°F. water of crystallization escaping from the fully hydrated crystals passes upward through the mass as a vapor, and, wherever it may happen to condense on cooler particles of the salt, is reabsorbed by partially hydrated crystals to form the completely hydrated salt, and in so doing unites with other fully hydrated crystals to form masses of considerable size.

The author is inclined to believe that Mr. Monfort's explanation is correct, but that in the drying out of the crystals in the course of manufacturing the lower hydrate may be formed in considerable amount at times, and that the rehydration of these crystals with water derived from moisture in the air is frequently the source of the required water for this recrystallization and the cause for the cementing together of the mass. Caking will occur in bags stored for a considerable length of time in an open room

Lime Handling Plant at Columbus, Ohio.—The method of handling lime for the water purification plant at Columbus, Ohio, is similar to that at St. Louis, except that the plant is somewhat smaller and less elaborate.

Bin.—The equipment consists of a concrete bin (Fig. 95) elevated on columns over the railroad tracks located at the side of the head house of the plant. The bin is of concrete 25½ ft.

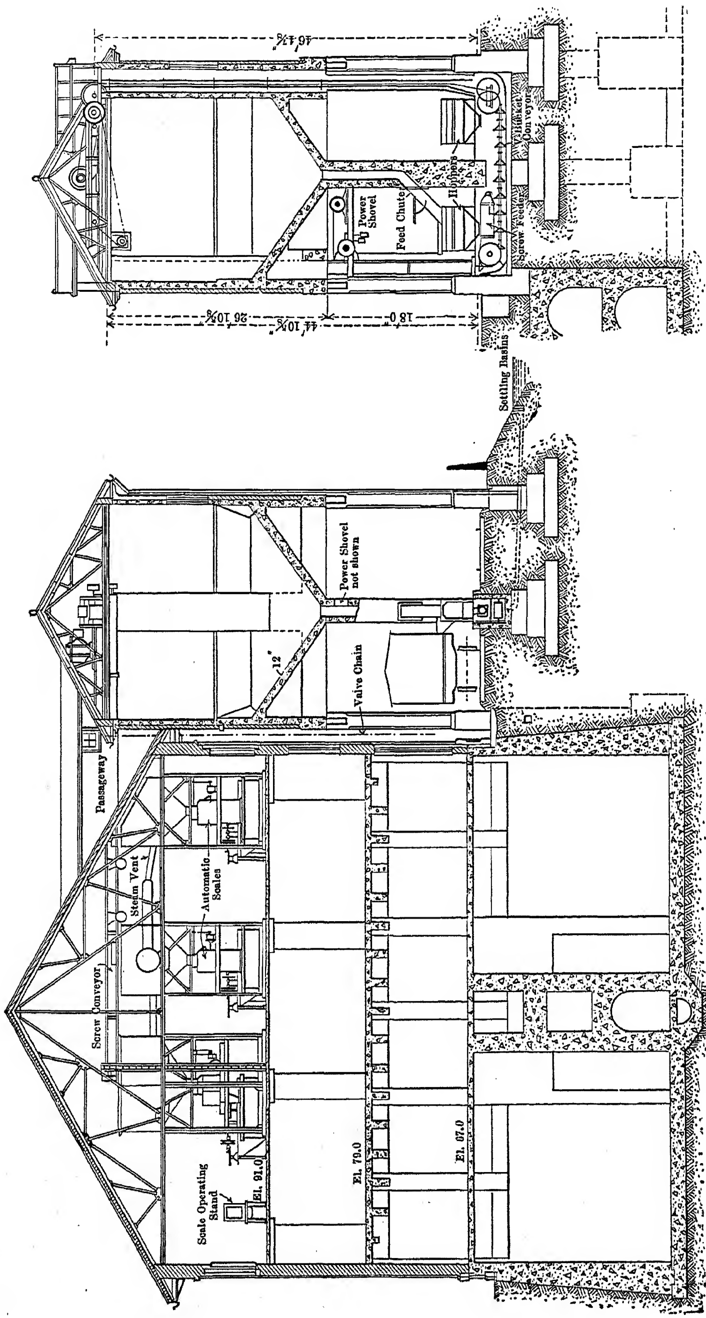

FIG. 95.—Section of lime-storage house at the Columbus, O., filter plant.

square with bottom slopes of approximately 38° with the horizontal, and heavily reinforced. The center column of the eight columns which support the structure, is cored, and is utilized as a chute to reclaim lime from the storage bin and deliver it by means of an elevator and conveyor to the tanks in the head house. The bin has a storage capacity of 220 tons of lime, or about 22 days' supply at the usual rate at which lime is used.

Unloading Lime from Cars.—A power shovel is used for unloading the lime from the cars into the hopper of the conveyor. It is operated by a separate 5-hp. 220-volt direct-current electric

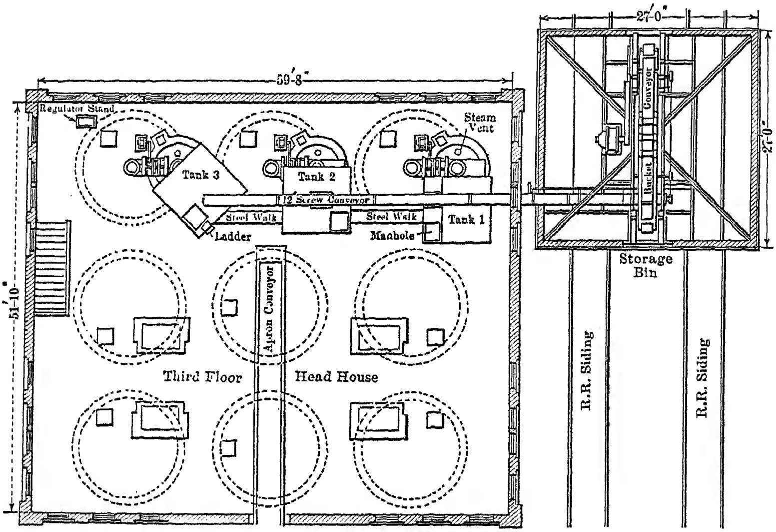

FIG. 96.—Plan of tanks and conveyor system used at Columbus, O., filter plant.

motor. Two men are required to manipulate the shovel and with it a 40-ton car of lime has been unloaded in about 3 hr.

Conveying Equipment.—This apparatus (Fig. 96) consists of a continuous V-bucket conveyor and elevator with a capacity of 50 tons per hour. It runs in a trough between the tracks and through chutes each side of the bin, being enclosed throughout its entire length. The lime is discharged from the elevator into a 12-in. screw conveyor, which latter carries it to the top of the steel supply tanks holding all together about 22 tons or 2 days' supply.

The lime from these supply tanks is fed into automatic weigh-

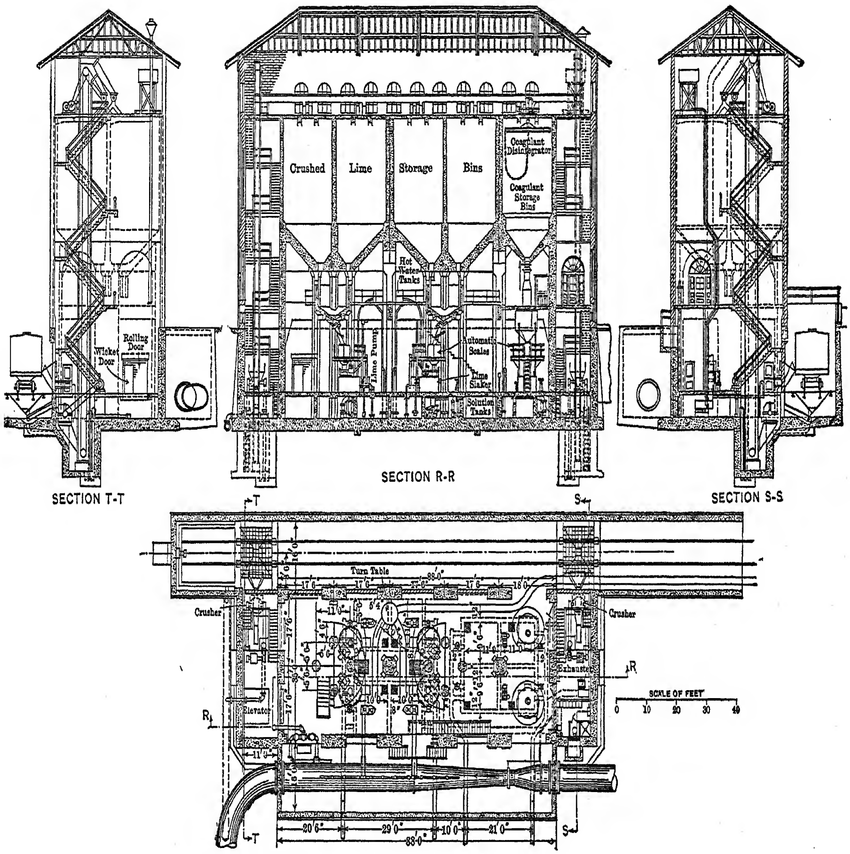

FIG. 97.—Coagulant house of the Cleveland, O., filter plant.

(Facing page 264.)

locked to prevent tampering with the equipment when once adjusted for any desired dumping interval.

MIXING AND STORAGE TANKS, AND PIPING FOR CHEMICAL SOLUTIONS

Tanks.—The tanks (Fig. 97) for preparing and storing chemical solutions are made of wood, iron and concrete. The sulphates of iron and aluminum may be dissolved and kept in concrete tanks, and will cause little deterioration in this material unless appreciable quantities of free acid are present. In the smaller plants wooden tanks are quite commonly used for these chemicals. Hypochlorite of lime solutions are best kept in concrete tanks, although wooden tanks lined with Portland-cement mortar will last a number of years if the lining is kept intact. Iron tanks lined with concrete are also used for this chemical, as well as lead-lined wooden and porcelain-lined iron tanks. Acid-resisting paints for both wood and iron tanks are also used. Soda ash should be dissolved and kept in iron tanks as it attacks concrete.

Lime should be slaked in iron tanks, but may be stored, if cold, as a milk of lime, or as a lime-water solution in concrete tanks. The heat generated by the slaking of lime in concrete tanks is liable to produce cracks in them, and thus cause them to leak and become useless.

The capacity of chemical solution tanks should be properly proportioned to the total output of the plant. The method of applying the chemical solutions will affect the size of the solution tanks, as this will depend upon whether they are used as real supply tanks which hold a definite volume of a standard solution, or whether they serve as dissolving tanks through which flow a constant volume of solution of variable but controlled strength.

The character of the coagulating chemical used also plays a part, since the relative solubilities of the various chemicals employed differ widely. The slight solubility of lime, for example, requires that large tanks be used if a real lime-water solution is to be applied to the raw water. On the other hand, if the chemical is used as a milk of lime, relatively small tanks may be employed. The slight solubility of bleaching powder in water makes it necessary to provide fairly good-sized tanks for

the storage of its solutions, even though the quantity of the solution used is quite small.

In those plants where solutions of standard strength are measured through orifices, there should be sufficient tank capacity to provide treatment of the water for 8 to 12 hr. without preparing new solutions. Never less than two tanks should be available for holding any one chemical solution, and in large plants three or four units are usually provided.

Mixing Tanks.—The rate of solution of a chemical compound in water is affected by the size of the solid particles, by the amount of agitation to which they are subjected, and usually to the temperature of the water used. Chemical reactions with the water may also take place, as for example in the hydration of lime. All these factors have to be taken into consideration in providing the best method of treating the chemical to be dissolved.

Stirring Apparatus.—The rapid solution of the solid chemical compound and the production of a uniform solution, as just noted, is facilitated by stirring, and this is usually done in small plants by hand with a paddle, but is preferably effected mechanically. Rotating paddles furnish as simple a means as any for aiding solution, and are effective. Paddles may be driven by an electric motor or by a small waterwheel. If a waterwheel is used the water driving the wheel may be discharged into the solution tank, and thus aid in the preparation of the solution. Compressed air is frequently used to produce a mixing action in solution tanks by being blown in through perforated pipe at the bottom of the tank.

Mixing Device Used in Cincinnati Filter Plant.—A mixing device used for aiding the solution of sulphate of iron in the Cincinnati filter plant consists of a small-bladed propeller driven by an electric motor and rotating in a pipe open at both ends and submerged in the center of the solution tank. Water is fed into the tank through a perforated pipe at the bottom, and the solution that is formed overflows at the top of the tank through an opening into a screen compartment connected to the chemical-solution pipe line. The solid sulphate of iron is dumped through a grating into a branch of the pipe inside of which the propeller is rotating, and falls through the propeller blades and pipe to the bottom of the tank. As only 100-lb. charges are dumped into the tank at any one time, solution is rapidly effected, and

takes place in part while dropping through the pipe and tank. The propeller blades act in a measure to lift the solution over the top of the submerged pipe, and in so doing draw liquid from the bottom of the tank and thus keep up a circulation between the top and bottom of the tank.

Lime Slaking.—The preparation of milk of lime requires a somewhat different stirring apparatus than those used in dissolving alum, copperas and soda ash. Where caustic lime is used it must be slaked, that is hydrated. This is carried out in

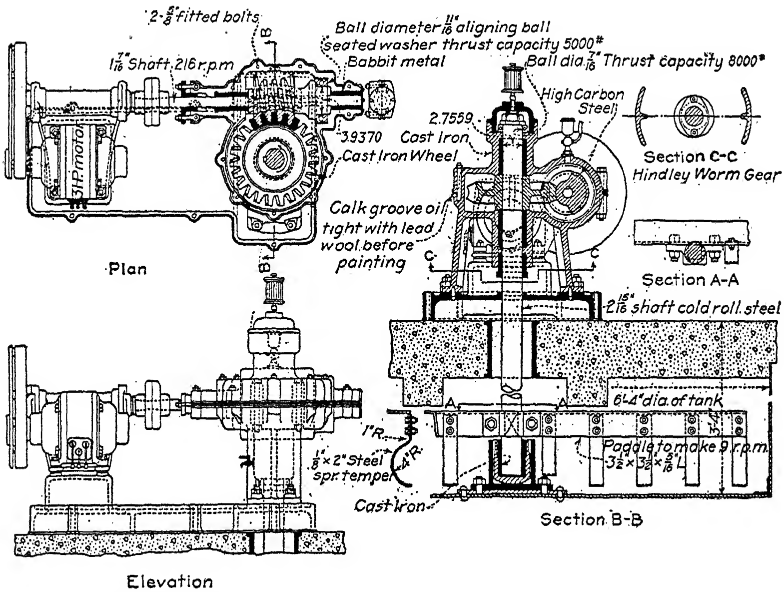

FIG. 98.—Details of stirrer for lime slaking designed for Jerome Park filter plant in New York City.

steel tanks fitted with revolving rakes, which keep the solid lime and water in contact until hydration is complete. A typical design for such a stirring apparatus is shown in the accompanying cut (Fig. 98).

Ordinary concrete mixers have been used for preparing milk of lime. This method of lime slaking is used in the water-purification plant at Grand Rapids, Mich., and at Baltimore, Md.

Solution of Bleaching Powder.—For mixing solutions of bleaching powder (Figs. 99 and 100) there are a number of devices on the market, all of which aim to get as intimate a

mixture of the powder and water as possible. As soon as the powder is wet it becomes pasty and lumpy. These pasty lumps must be rubbed up with water if a good solution is to be made. An accompanying cut shows how this is done in one form of mixer.

The disagreeable gases arising from solutions of calcium hypochlorite while being prepared led to an ingenious design to overcome this difficulty in connection with the plans for the Grand Rapids, Mich., filter plant. The sheet-iron drums of this chemical, weighing 700 or 800 lb. each were to be opened under water in a dissolving tank provided with a false bottom, consisting of cast-iron grate bars. The drums are lifted into position by means of a chain block. Sharp steel points are then driven into the head of the drum by means of a shaft extending through a stuffing box in the side of the tank. This shaft is forced home by means of a lever on the outside. By means of a ratchet and pawl the drum is turned while a large

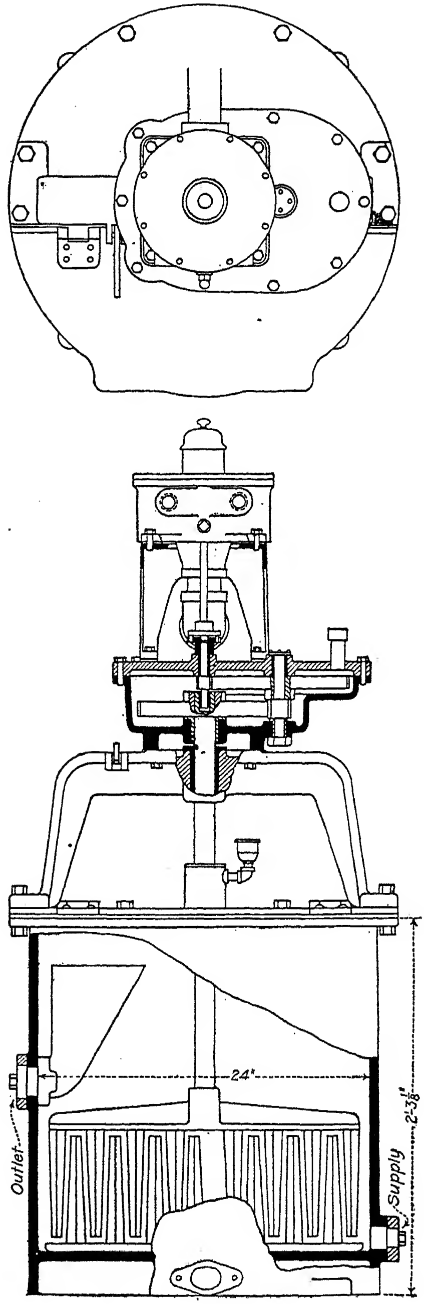

FIG. 99.—Details of a design for 24-inch motor-driven mixer used for dissolving bleaching powder.

can opener cuts the can in two. This operation is effected after the tank has been flooded with water. A similar device has been used at the filter plant in Minneapolis, Minn.

Piping, Valves and Pumps for Chemical Solutions.—The conveying of chemical solutions from the supply tanks to the point where they are discharged into the water to be treated presents some difficulties which are not easily overcome. All pipe lines for this purpose should be as short as possible and as straight as it is practicable to make them. Where bends in pipe must be made crosses or plugged tees should be used so that the line shall be straight between bends, and so that cleaning points are provided. All chemical-solution pipe lines ought to be so placed as to be easy of access, both for cleaning or temporary removal. As a rule, they should not be laid under ground, nor buried in concrete or masonry.

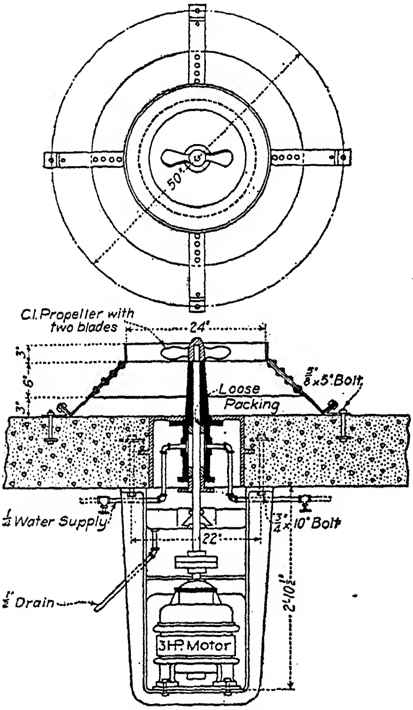

FIG. 100.—Details of agitator for bleaching-powder solution tank designed for Montreal, Quebec, filtration plant.

As incrustations and deposits are likely to form in pipes conveying certain of these solutions, the pipes should be of sufficient size so that the lessening diameter of the pipes, as incrustation proceeds, shall not render them inadequate for carrying the necessary volume of liquid with the pressures available. On the other hand, if pipes are too large the diminished velocity of the flowing liquid may only hasten the formation of deposits and the ultimate clogging of the pipes. The effect of corrosive liquids can be overcome only by the use of pipe material which is not attacked by the liquid to be handled.

All chemical-solution lines should be provided with means for flushing them out with water under pressure. In some cases

steam and hot-water connections will also materially assist in removing certain kinds of deposits. Certain deposits, as for example those formed in pipes conveying milk of lime or lime-water solutions, can not be removed by flushing and must be scraped out with scrapers. Deposits of lime that have been accumulating for some time in pipes, are very difficult to remove, and not infrequently involve the removal of the pipe entirely and the substitution of new pipe in its place.

For conveying sulphate of alumina solutions (Fig. 101), lead, special bronze or hard-rubber piping are the best to use. Lead and brass or bronze piping are probably slightly attacked by this solution, but are usually found satisfactory where good grades of the chemical are used. Hard rubber is not attacked at all,

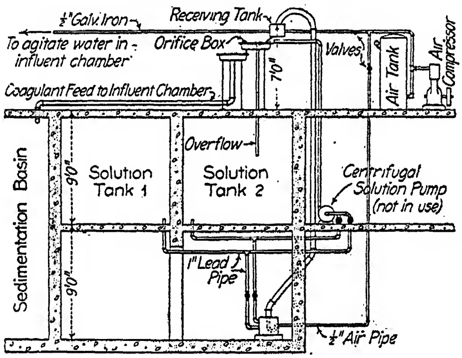

FIG. 101.—Airlift for alum solutions.

but is liable to crack, beside being expensive. Iron pipe may be used for lime and soda-ash solutions, as it is not attacked by these chemicals. Sulphate of iron solutions, if not too acid, may be safely run through either cast or wrought iron, or even steel pipes. Slightly basic sulphate of iron tends to deposit iron hydroxide in pipe lines, and its corrosive action is very slight. This is not the case, however, if appreciable amounts of free acid are present.

Calcium hypochlorite solutions may be conveyed through iron pipes, but both corrosion and incrustation occur, and the life of the iron pipes is short. Tile, lead and hard-rubber piping may be used for conveying this solution, as none is attacked. Lead-lined iron pipe, brass, bronze and copper pipe have been used, as well as ordinary rubber hose.

Dry chlorine gas may be conveyed through iron or brass piping without corroding them. If water either as a liquid or as a vapor comes into contact with the gas while in the pipe, corrosion will result. Lead is probably attacked less than most metals, and is used to deliver the gas into the water to be treated. Hard-rubber or tile pipe is also used for discharging the gas under water, and also small pipes made of silver. Ordinary rubber garden hose has been successfully used by the author, but in time the rubber is attacked and becomes brittle and breaks. However, the low first cost of the hose makes its renewal easy and inexpensive.

In general, the fittings and valves for chemical-solution pipe lines are of iron, brass, special bronzes or hard rubber. Lead, copper, stoneware and glass valves have been used, but more especially for corrosive solutions like those of bleaching powder. Gate and globe valves are commonly employed, and sometimes brass cocks. The latter, however, are not as a rule desirable on account of their liability to stick.

Pumps.—Small pumps of both the reciprocating and centrifugal types are used for forcing chemical solutions through pipe lines to some higher elevation or into the water to be treated. They should not be used if it is possible to avoid it, since they are, of course, subject to the corroding action of the liquids they handle with all the attendant difficulties. Pumps of hard rubber or of Monel metal are practically proof against corrosion, but are expensive. Pumps with cast-iron pump ends lined with lead are not infrequently used for the more corrosive solutions.

Eductors.—Mr. George W. Fuller recommends the use of water eductors for handling chemical solutions. He states:

"They have the double advantage of being of ideal simplicity of form, and of providing additional diluting water to the solution in its passage through the pipes, with a consequent reduction in the objectionable action of the chemicals on the piping. The proportions of the eductors can be varied practically at will for handling solutions against different heads and with different working water pressures. The relations between these variables can be expressed by the formula:

$$\frac{q}{Q} = \sqrt{\frac{H}{h}} - 1$$

where q equals the quantity of solution to be pumped, h the head to which the solution is to be pumped, Q the quantity of pressure water

used, and H the available head of the pressure water: thus if the pressure water is available at 200 ft. head, and the solution is to be pumped against a head of 40 ft., 1 volume of pressure water will handle 1.22 volumes of solution."[1]

These ejectors can probably be used satisfactorily with chemical solutions that do not tend to incrust. With milk of lime they have not proven entirely satisfactory.

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CHAPTER XXII

APPARATUS AND METHODS FOR APPLYING CHEMICALS AND THE PREPARATION OF SOLUTIONS

Apparatus for Feeding Dry Chemicals.—The application of chemicals to a water should be obviously regulated with considerable care in order tc prevent either under or overtreatment. The application of the solid chemicals in a dry state directly to the water to be treated is used to a limited extent. The method possesses undoubted advantages in eliminating standard solutions of definite strength and the large tanks necessary to store them; and also in doing away with all apparatus for measuring their rate of application. On the other hand, the physical characteristics of some of the chemicals used in water purification, and the imperfections of much of the apparatus that has been designed for feeding dry chemicals, have prevented the general adoption of the method.

In spite of the mechanical difficulties which the method of dry feeding presents, an apparatus designed by W. Donaldson in 1910, in which friction drive of a worm gear actuated a feed screw and permitted a nice adjustment of the weight of the substance through a wide range, was designed and is used in seven plants at the present time, eight feeders being used for hydrated lime and one for sulphate of iron. This particular device did not work well with soda ash.

Dry-feeding apparatus (Fig. 102) usually works well with aluminum sulphate as it is not hygroscopic and does not cake and bridge over the orifice. Hydrated lime may be fed in this way with considerable success in spite of its tendency to bridge over an orifice. This latter difficulty occurs with any fine amorphous material, and of course causes intermittent feeding. It is practically impossible to feed bleaching powder successfully in dry-feed apparatus.

Mr. Allen Hazen in conjunction with Mr. R. S. Weston has designed a dry-feed apparatus that has been successfully used for several years to feed crushed sulphate of alumina to a water,

and it is claimed that experimentally the apparatus with some modifications has been also successfully employed in applying calcium hypochlorite.

The Hazen apparatus consists of "a screw conveyor operated in the bottom of a large hopper containing the dry chemical in such a state of physical subdivision that it would be steadily fed." Motive power for driving the conveyor is furnished by an electric motor or by a Pelton waterwheel. If the latter is used, the water discharged from the wheel may be employed in the dissolving box into which the dry chemical is dropped. The speed of the screw is controlled by a system of multiple gearing, which permits

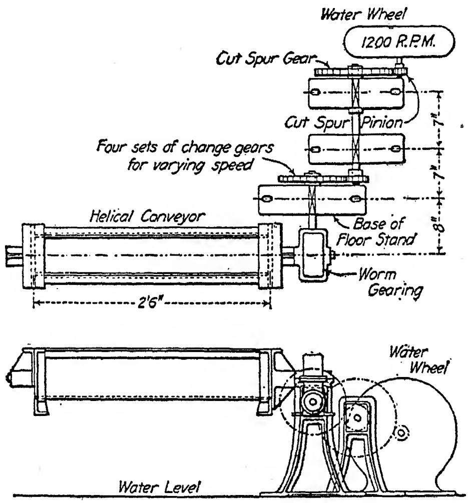


FIG. 102.—Dry-feed chemical apparatus.

of a wide range in speed between the motor and screw, and by changing the speed of the motor.

In apparatus installed in plants at Springfield, Mass., and in Poughkeepsie, N. Y., a 4-in. screw driven by a variable-speed motor with a range in speed from 1 to 4 for the motor, and with gears with a range from 1 to 56, enables one screw to feed chemicals with a variation in speed from 1 to 224, a range considerably in excess of the variations likely to be required in any plant. A 2-in. conveyor will discharge, according to the gears in use from 1 to 3 tons of coagulant per day, or the equivalent of 1 grain per gallon to as much as 40,000,000 gal. of water; while on the other hand, it may be run slowly enough to treat as little as

100,000 gal. of water per day. A revolution counter attached to the screw conveyor provides a method for keeping a record of the amount of chemical used. The screws which have been employed in this type of apparatus ranged from 2 to 4 in. in diameter, but the preference would seem to be for a screw of small diameter and operated at a relatively high speed. Tests of this apparatus have shown that the discharge of chemicals per revolution varies less than 3 per cent. from the lowest to the highest speed.

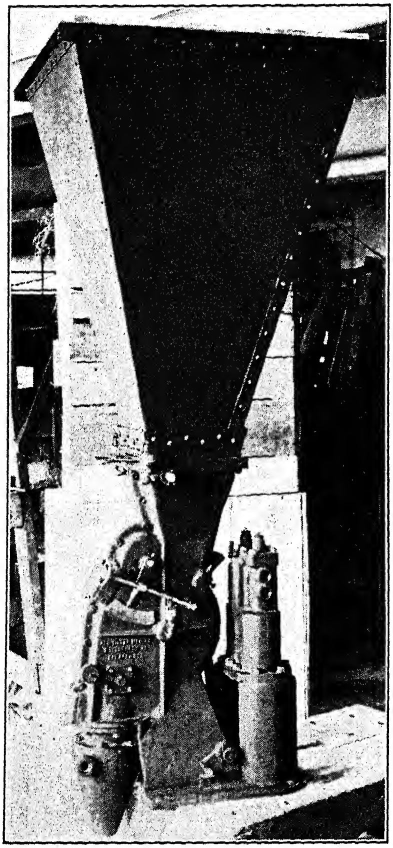


FIG. 102a.—Dry-feed chemical apparatus (Pittsburgh Filter Mfg. Co.).

To effect a rapid solution of the chemical it should be dropped into flowing water. By keeping the solid particles in motion until they are dissolved, the settling out of the solid may be prevented.

In Figure 102a is shown the cut of a dry chemical feeding machine[1] which has been used successfully in feeding hydrated lime at the Toledo, Ohio filtration plant. This machine consists of a feed hopper with an agitator, a revolving drum with a ratchet drive, an adjustable orifice, a mixing compartment and a motor for driving the drum. The motor used was driven by water under pressure, and the discharged water was used for washing the lime into the pipe line through which the lime was being applied to the raw water. By adjusting the stroke of the motor, and the aperture of the orifice, it was possible to obtain a fairly uniform feed. The rated capacity of the machine varied from 90 to 14,000 pounds in 24 hr.

[1] Made by the Pittsburgh Filter Mfg. Co., Pittsburgh, Pa.

Modification of the Method of Dry Feeding of Chemicals. St. Louis Method.—In 1904 a method of applying chemicals to the water supply of St. Louis, Mo., was tried, which proved to be quite successful. By this method the chemicals were weighed out in small quantities and dumped by hand at regular intervals into solution tanks. These tanks were small and the solution formed by the discharge into the tanks of a sufficient but unmeasured volume of dissolving water, overflowed continuously into the water being treated. No accumulation of undissolved chemical was permitted, and the amount dissolved was, therefore, equal to the amount applied, which latter was, of course, adjusted to the quantity and character of the water being treated.

The chemicals, which were used, were sulphate of iron and caustic lime. The so-called "sugar sulphate of iron" was employed on account of the ease with which it dissolves and because of its purity. Weighed quantities of the iron sulphate were dumped at regular intervals into a wooden dissolving box 4 ft. 8 in. by 5 ft., by 3 ft. deep inside, and which was supplied with water through 20 pipe nipples, 3/8 in. in diameter, and closed with brass caps. Each cap had four 3/64-in. holes drilled in the side, and so placed that the issuing water made a fairly uniform sheet over the bottom of the tank. The solution escaped through overflow pipes near the top of the tank. The solution was fed through wrought-iron pipes directly into the uptake shaft through which the water was passing from the intake to the wet well of the low-service pumping station. With some few modifications the same apparatus and method are still followed in the new coagulant house previously described.

The lime-slaking apparatus consisted of four tanks into which the weighed lump lime was dumped from wheel-barrows at regular intervals. A constant volume of water at a temperature of about 120°F. was fed into these tanks. The milk of lime formed had a temperature of 140° to 180°F., and overflowed into the uptake shaft, the same as did the sulphate of iron solution. On account of the lime incrustations on valves and pipes, this point of application of the milk of lime was abandoned, and centrifugal pumps were installed, which forced the liquid to a receiving well beyond the pumps and between the latter and the settling basins.

The stirring apparatus for each lime-slaking tank consisted of a vertical shaft with a crossarm about 1 ft. from the bottom of the

tank, and provided with rakes. The rake had the form of a half-circle, swinging in a vertical plane about the crossarm and with an arrow head on the bottom end. A 5-lb. weight fastened at the back of the rake gave the necessary resistance to swinging and brought it back to position if temporarily raised by striking lumps of lime which would cause it to jam, while still holding it in place to move everything in front of it.

This apparatus has been somewhat modified in the new coagulant plant of the St. Louis Water Department, although the general principles involved are the same. The slaking tanks are of steel instead of wood, the stirring apparatus is more strongly built, and is driven from below the tank from a single line shaft. An ingenious arrangement of the vertical shaft from the bevel-gear drive avoids a stuffing box in the bottom of the tank. The tops of the tanks are covered. They are designed to slake 80 lb. of lime per minute, but can readily slake 90 lb. per minute.

In the old plant the volume of slaking water added was 30 gal. per minute per tank, irrespective of the amount of lime added. This made the maximum strength of the milk of lime 8⅓ lb. of water to 1 lb. of lime, or 7,000 grains of lime per gallon. The present method is to add about 3¼ lb. of water to 1 lb. of lime. The resulting temperature of the milk of lime is about 200°F. As this liquid has to be cooled before it could be pumped, a steel heater tank was installed to utilize this heat to raise the temperature of the water used to slake the lime, and thus reduce the quantity of steam formerly used to heat the slaking water. The heater tank is 4 ft. 6 in. in diameter and 3 ft. 3 in. in depth, and contains 1½-in. copper pipe coils, and a stirring apparatus. The milk of lime from all the slaking tanks discharges into the top of this tank, and it in turn discharges the cooled milk of lime through an overflow pipe near the top of the tank into the pump box, where, after dilution with cold water, it is forced by pumps to the delivery well.

The efficiency of this method of slaking lime was found by Mr. W. F. Monfort to be considerably affected by the initial temperature of the water. In the following table he shows a series of results obtained in experiments made with this apparatus.[1]

By maintaining an initial temperature in the hot-water supply of 160°F., and a temperature of 200°F. or more in the slaking

[1] *Eng. Record*, vol. 56, July 27, 1907.

Series	Tank	Initial temp., deg. F., milk lime	Maximum temp., deg. F., milk lime	Efficiency, per cent
A ..	1	110	156.0	70 30
	1	120	171.3	78.13
	4	122	156.0	72.85
	3	172	202.0	83.65
B	4	127	165.0	66.04
	3	165	200.0	81.25
C	4	117	153.5	79.35
	3	172	208 0	94.60

tanks, Mr. Monfort concludes that a 15 per cent. greater efficiency can be obtained in slaking the lime.

At the Cincinnati filtration plant, where the same general methods of applying chemicals have been followed as at St. Louis, a comparatively weak milk of lime is used. The strength of this milk of lime will range from 1,000 to 2,500 grains per gallon, and will average about 1,500 grains per gallon. The water used to slake the lime is heated to a temperature of 140°F. The milk of lime is discharged directly into a 40,000-gal. tank through which settled water is constantly flowing at a rate of about 900 gal. per minute. The milk of lime enters with the water flowing in at the bottom of the tank, and becomes dissolved as it flows upward with the diluting water, and passes out over the weirs at the top of the tank. A slowly moving stirring device rotating at the bottom of the tank assists in diffusing the lime through the water and thus aiding in its solution.

In this way there is produced a lime-water solution, which will have a strength of approximately 30 grains per gallon. The strength of this lime-water solution will, of course, vary, depending upon the time interval at which 100 lb. of the ground lime is dumped into the slaking tank. It is evident that the quantity of lime being slaked for a single lime-water saturator tank must not exceed that amount of lime which the water passing through the saturator is able to dissolve, viz., about 60 grains of calcium oxide per gallon.

The efficiency of this method of applying lime was shown by a series of tests made by the author to be about 92 per cent. The results of these tests as tabulated in Mr. George H. Benzenberg's report as chief engineer of the Commissioners of Water Works of Cincinnati, Ohio, are as follows:

	Pounds	Per cent.
1. Soluble lime in lime water applied to raw water...	54,728	77 4
2. Soluble lime used up by the raw water required for making lime-water solution	2,834	4 0
3. Soluble lime in dry sludge taken from slaking tanks.	1,755	2.5
4. Insoluble material in sludge from slaking tanks...	6,229	8.8
5. Insoluble material presumably carried into saturator tank with milk of lime	5,154	7 3
Total	70,700	100.0

Percentage of the total available soluble lime in lime-water solution applied to raw water..... .. . . 92 2

Average strength of lime-water solution equaled 28.8 grains of CaO per gallon.

APPLICATION OF MEASURED VOLUMES OF CHEMICAL SOLUTIONS OF STANDARD STRENGTH

The more common method of applying chemicals to a water to be treated is by means of an orifice or weir, through or over which flows a standard-strength solution of the chemical to be applied. The volume of this solution must, of course, be regulated so as to be properly proportioned to the volume of flow of the water being treated. This is effected usually by varying the size of the orifice or the width of the weir, although in some apparatus it may be produced by a change in the head coupled with a change in the size of the orifice or weir.

Orifice Feed Tank.—A typical orifice feed tank is shown in the accompanying cut (Fig. 103).

It consists of a cast-iron porcelain-lined tank, into which the chemical solution is fed from a large storage tank. The solution enters through a cock controlled by a glass float. By means of the ball cock and float a uniform depth of solution is maintained in the orifice tank. An overflow pipe of hard rubber is shown in one corner of the tank.

The orifice through which the solution escapes is located near the top of a vertical pipe of hard rubber which passes through a rubber stuffing box in the bottom of the tank. This pipe may be moved up and down by means of a hand wheel and threaded stem, and thereby submerge the orifice to any desired depth. The depth at which the orifice is placed is indicated directly on a dial. The threaded stem has 16 threads to the inch, and one revolution of the hand wheel corresponds to a movement of the pipe, and consequently of the orifice, of 1/16 in. Very accurate

adjustments of the depth of the orifice is, therefore, possible. Placing the orifice in the vertical feed pipe, so that it is submerged and yet is above the bottom of the feed tank, diminishes the chances for its becoming clogged by floating or deposited material.

Where it is desired to apply the chemical solution to suction pipe lines conveying water to the pumps, it is necessary to use a suction tank in connection with any gravity feed orifice or weir tanks, in order to prevent entrance of air to the pump suction

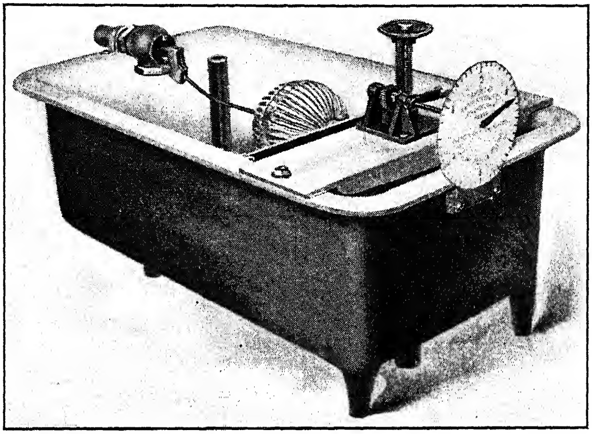


FIG. 103.—An orifice feed tank.

pipes. This is effected by discharging the chemical solution from the orifice feed tank into an intermediate tank directly connected with the suction pipe. By means of an auxiliary supply of water entering through a cock controlled by a float, this intermediate tank is kept filled with water, thus sealing its outlet to the atmosphere and preventing air entering the suction pipe of the pump.

Many variations from the general type of orifice feed tank described above are in use. Rectangular and circular orifices with adjustable gates are commonly employed, and automatic adjustment of the openings to correspond with changes in the volume of water being treated is not unusual in the larger plants.

Proportional Chemical-solution Feed Pumps.—The variation in the flow of water through a purification plant, in order to meet fluctuations in consumption, necessitates a proportional variation in the dose of chemicals to be applied. This change

may be made automatically or by hand. In the smaller plants proportional chemical-solution feed pumps attached directly to reciprocating steam pumps, and actuated by the latter are sometimes used.

The front and rear head of one water end of the steam pump is drilled and tapped for pipe connections, which lead to the top and bottom, respectively, of the water end of the chemical pump. Each stroke of the steam pump causes a corresponding stroke of the chemical feed pump. A change in the amount of chemical solution to be added is effected by lengthening or shortening the stroke of the plunger in the chemical end of the feed pump. For any given setting of the pump the quantity of chemical applied is proportional to the water being pumped by the steam pump.

The motor end of the feed pump may be of brass. The chemical end is usually of iron, which for corrosive chemical solutions is lined with lead.

Control of Application of Chemical Solutions by Means of Venturi Tube.—Automatic regulation of chemical feed by means of the change in velocity of the flow of water to be treated as it passes through a constricted pipe is one of the more common methods employed, especially in the larger plants. The general principle involved in this method of control is clearly shown by the accompanying cut (Fig. 104).

This apparatus as described and tested by Mr. Ralph Hilscher, the patentee, depends for its action directly on the decreased static head at the restricted section of a Venturi meter, which is measuring the water to be treated. It consists of a tank divided into two compartments by a wall extending nearly to the top of the tank. Resting on the top of this dividing wall is a pivoted horizontal arm, from which are suspended two floats of equal dimensions, one in each compartment, and at equal distances from the pivot. The arm in compartment *B* extends beyond the float, and connects with a balanced valve on the end of the feed pipe, which supplies the chemical solution from a storage tank. The valve is made so that when the float *B* falls below the float in *A*, the downward movement of the arm will cause the valve to open and admit chemical solution to compartment *B*.

Compartment *B* is directly connected to the reduced section of the Venturi tube. It contains a valve which may be adjusted.

The full section of the Venturi tube above the constriction is connected directly to compartment *A*. When a flow of water occurs through the Venturi tube, unequal pressures at the full and reduced sections will tend to make the levels in *A* and *B* different, that in *B* being lower. This difference in head (*h*) is proportional to the square of the rate of flow through the Venturi tube, and, if utilized to force the chemical solution

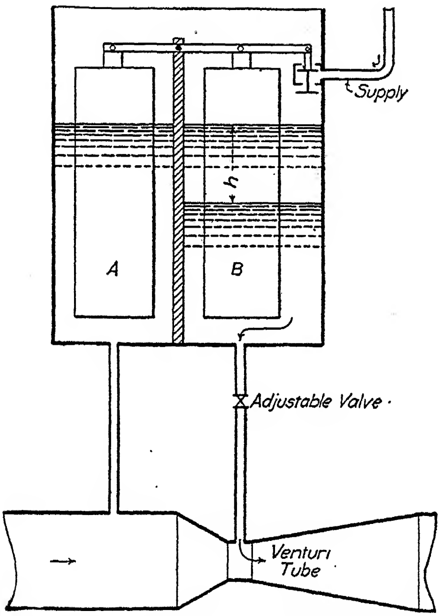


FIG. 104.—Diagram of chemical feeder.

through an orifice, will produce a flow through that orifice directly proportional to the flow in the pipe line. The adjustable valve acts as an orifice in this device, and the desired effective head on it is established by building up the level in *B* equal to that in *A*.

In tests made of an experimental apparatus, in which the rates at which the water was treated varied by about 300 per cent., departures from the average ratio of the volume of water treated to the volume of chemical solution being applied ranged from 7.7 per cent. below to 6.2 per cent. above the mean, but averaged only 1.7 per cent. on either side of the mean.

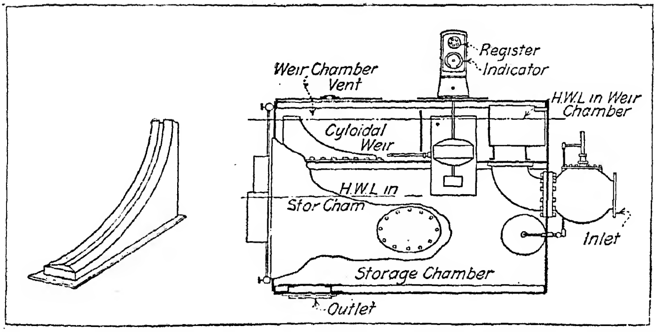


FIG. 105.—Proportional flow weir for feeding chemical solutions.

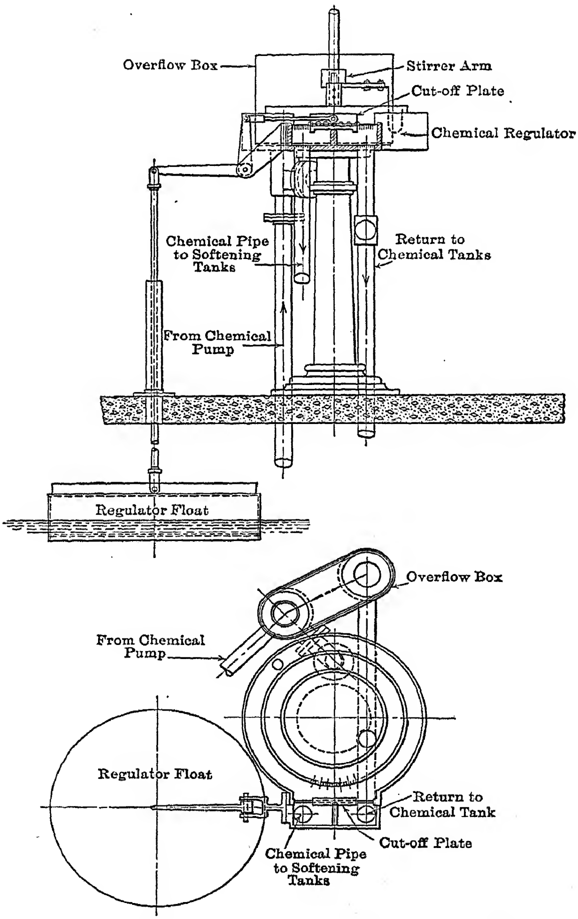


FIG. 106.—Regulation of chemical feed by means of Sutro weir

Control of Application of Chemical Solutions by Means of Weirs.—An ingenious application of this method of control is shown by the use of a proportional-flow weir for measuring the water to be treated in combination with an automatic adjustable weir for regulating the flow of the chemical solution. The Sutro weir[1] (Fig. 105) is of the proportional-flow type, that is the head of water on the weir is in direct proportion to the

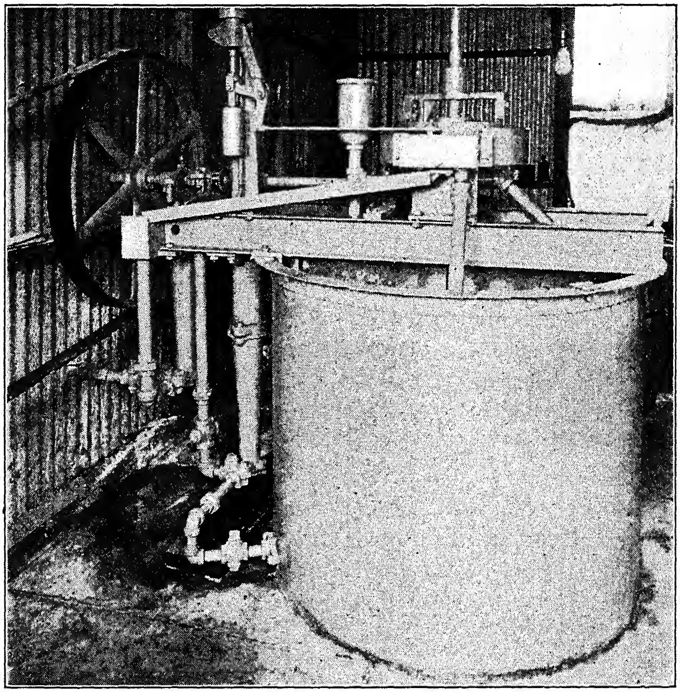


FIG. 107.—Booth's chemical regulator. (Made by the L. M. Booth Co.)

quantity flowing over it. By means of a float in a chamber connected to the weir box, the change in the volume of water to be treated is indicated by the rise and fall of the float.

This movement of the float is communicated by a system of levers to a cutoff plate, which varies the width of the weir over which the chemical solution is flowing. The head on the chemical-solution weir remains constant, as the cutoff plate merely diverts more or less of a constant volume of flow of the chemical solution to the discharge pipe, the balance flowing

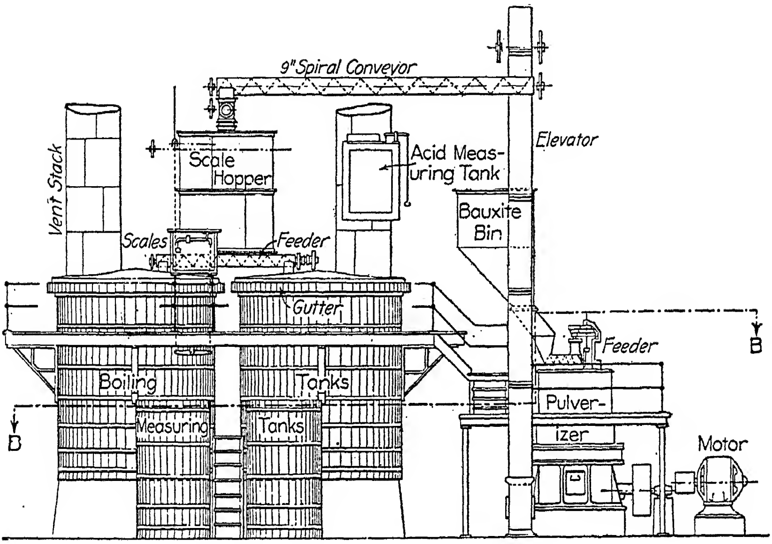
[1] Manufactured by the L. M. Booth Co.

through the return pipe to the storage tank. By thus varying the width of the chemical-solution weir, by means of this cutoff or diversion plate, while the head remains constant, it is evident that the volume varies in proportion to the lateral shifting of the plate, and consequently to the volume of water passing over the Sutro weir (Figs. 106 and 107).

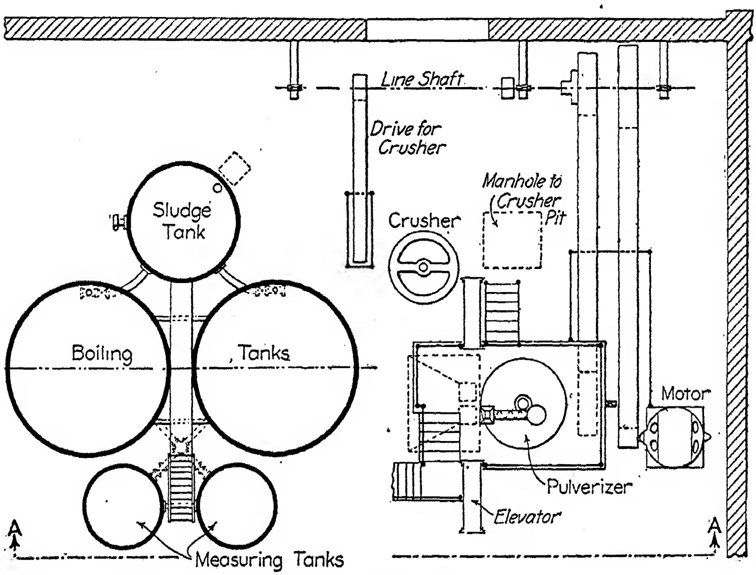
PREPARATION OF SOLUTIONS

A simple and effective method for preparing solutions of definite strength is to place in a compartment within and at the top of the solution tank, the weighed amount of the chemical compound to be dissolved. From a perforated pipe just above the compartment water may be sprayed over the chemical until it is dissolved, the solution overflowing into the large tank. By regulating the volume of the water used to dissolve the chemical, the large tank can be filled by the time solution is effected. Sulphate of alumina, sulphate of iron and soda ash are easily dissolved in this manner. Soda ash, if dumped into the bottom of a tank and solution attempted by covering it with water, tends to form a solid crystalline mass very difficult to dissolve. Sulphate of iron is quite easily dissolved, especially in the form of fine-grained crystals known as "sugar sulphate of iron." This salt has a tendency, however, to form hard masses of crystals at the bottom of a tank, when large quantities of the salt are being dissolved without adequate stirring.

At the Columbus, Ohio, water-purification plant, Mr. C. P. Hoover, chemist in charge, has perfected a process whereby an aluminum sulphate syrup (Fig. 108) is manufactured directly from bauxite and sulphuric acid at the plant. The bauxite and sulphuric acid are boiled together in lead-lined tanks until a basic solution of aluminum sulphate is obtained. The solution is then diluted with water and is measured as needed into alum solution tanks, in which it is further diluted with sufficient water to make a solution of the standard strength that is to be applied to the water being treated. By this process it is claimed that five distinct steps in the manufacture of aluminum sulphate and its preparation for use in water purification are eliminated, namely, filtering, concentrating, crystallizing, grinding and redissolving. It is estimated that a very considerable saving in cost is obtained as compared with the cost of using the solid aluminum sulphate, and that even greater reductions in cost can



Elevation A-A



Sectional Plan B-B

FIG. 108.—An alum manufacturing plant at the Columbus, O., filter plant.

probably be obtained by utilizing a cheaper raw product known as halloysite as a substitute for the bauxite.

The following is an estimate of the cost of producing 1000 tons of 17 per cent. Al_2O_3 alum in solution at the Columbus Plant:

Estimated Cost of Producing 1000 Tons of 17 Per Cent. Al_2O_3 Alum in Solution

468 tons 66°Bé. sulphuric acid @ $12.50	$5,850.00
265 gross tons bauxite @ $9.90	2,623 50
Lubricating oil	20.00
Steam	100.00
Electric current, 10,000 kw.-hr. @ 2 cts	200.00
Repairs to plant	500.00
Depreciation	600.00
Interest on investment	600 00
Total	$10,493 50

In preparing solutions of bleaching powder, the compound must be rubbed into a paste with water, and thoroughly agitated with the dissolving water, so that it is diffused throughout the latter in a finely divided form. The tendency of this material is to form pasty lumps, the interiors of which are not reached by the water used to dissolve them. The large amount of insoluble material always present, with the consequent formation of sludge in the bottom of the tank, adds to the difficulty in obtaining the full value of this chemical in aqueous solutions. Not more than 8⅓ lb. of bleaching powder (1 per cent.) should be added to 100 gal. of water in preparing solutions, and a better solution with less waste will be produced if only one-half this weight of powder is used to the above volume of water.

The strength of solutions of coagulating chemicals that are commonly used varies more or less with local conditions, such as tank capacity, volume and character of water to be treated. A convenient strength for a sulphate of alumina solution is 2 per cent., while solutions of sulphate of iron may vary from 2 to 5 per cent. Too strong solutions are undesirable on account of their greater corrosive and incrustation effects in tanks and pipe lines. The accumulation of sludge in chemical solution tanks should be avoided by frequent cleaning. The clogging of feed lines and drains with these deposits may be minimized, but not entirely avoided by frequent flushing with water under considerable pressure. In some cases hot water and even steam may be

found beneficial in clearing long lines of chemical feed pipes. In iron sulphate solution tanks and pipe lines, iron bacteria (Crenothrix and Leptothrix) have been found growing in such profusion as to form slimy sludges which adhere to the sides of the tanks and clog the pipe lines. The author's attention was called to these growths by Mr. W. F. Monfort of St. Louis, and he was able to confirm his observations by finding abundant growths of Crenothrix in the sludge of sulphate of iron solution tanks at the Cincinnati filtration plant.

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CHAPTER XXIII

POWER PLANT, PUMPING MACHINERY, AIR COMPRESSORS, AIR TANKS, WASH-WATER TANK AND MISCELLANEOUS EQUIPMENT

A properly equipped rapid sand filter plant requires more or less power in order to operate its pumps, air compressors, stirring devices, and other auxiliary apparatus. The location of a filter plant near a pumping station often makes it possible to obtain all the necessary power from the station. On the other hand, a filter plant may be so located that an independent power plant is necessary. The larger the filter plant the more desirable does an independent power plant become.

Power Plant.—Where power is derived from a nearby pumping station which is operated by a steam power plant, it may be possible to pipe the steam under pressure directly to engines driving the pumps, air compressors and other machines to be operated in the filter-plant buildings. It is generally easier, however, to make use of the steam at the pumping station to drive electrical generators, and to convey the current thus formed to electric motors, which can be used for operating all the machinery used in the plant. For short distances a direct current may be used without a too great cost for conductors; but for long distances an alternating current will be found the cheaper means for conveying the power.

Because of the ease with which power may be conveyed by an electric current, it lends itself admirably to nearly all of the needs of filter-plant operation. Wash-water pumps, air compressors, stirring devices, valves, refrigerating apparatus, and other devices for which motive power is needed, can all be handled with electric motors. Even low-service centrifugal pumps for supplying untreated water to the purification plant, and which in some cases may be regarded as almost an integral part of the filter plant, are usually motor-driven. Plunger pumps also, when provided with proper gearing, may be motor-driven, and are frequently used.

Where hydraulic power is available it furnishes a means for generating electric current at a low cost. Even comparatively low heads of water may be utilized, provided a sufficient volume of water is available. Even where the water power is not always sufficient, and must be supplemented at times by steam-driven units, the usual low cost of current obtained from generators driven by water power makes it well worth while to employ it wherever possible.

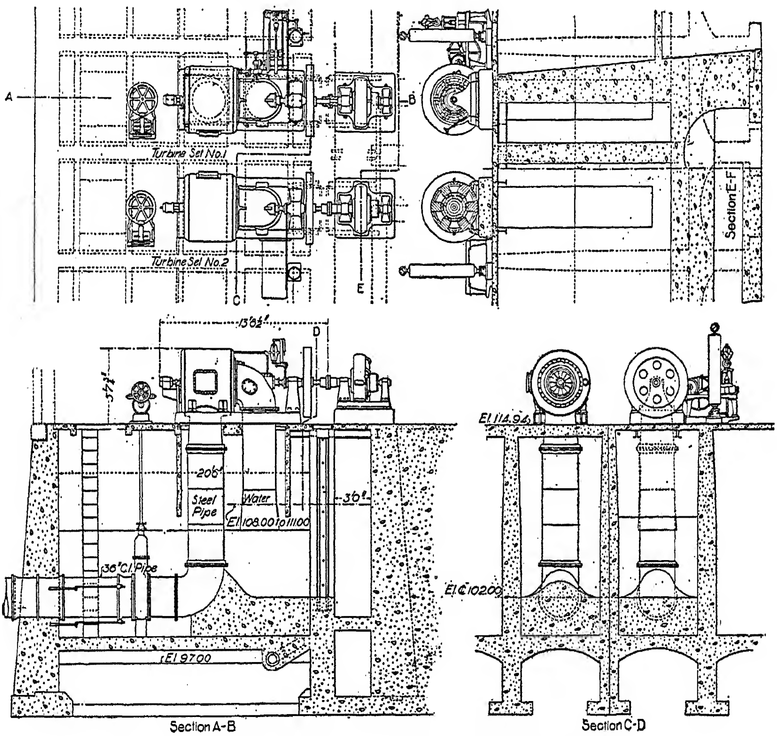

Fig. 109.—Water wheels at the Cincinnati filter plant.

Power Plant at Cincinnati Water-purification Plant.—A somewhat unique utilization of water power (Fig. 109) is successfully employed at the Cincinnati filtration plant, where the water to be purified is drawn from settling and storage reservoirs to the head house of the filter plant. The available head of water is about 28 ft. The water from the settling reservoirs flows through two 60-in. pipes to the front of the head house. Six 36-in. pipes connect the two 60-in. mains with concrete channels in the base-

ment of the head house. On each 36-in. pipe line is a motor-operated gate valve, by which the flow into the head house is regulated.

On each of three of these 36-in. pipes there has been installed a Trump horizontal waterwheel. Each wheel is rated at 83 hp. with a flow of 2,100 cu. ft. per minute, or when about 22,000,000 gal. of water passes through each wheel in 24 hr. The waterwheels are connected by flexible couplings to Fort Wayne 50-kw., 230-volt direct-current generators, and are operated at 360 revolutions per minute. The speed of each wheel is controlled by a Lombard type M oil-pressure governor. Speed regulation is also assisted by a cast-steel flywheel placed on the turbine wheel shaft, and which weighs 3,900 lb.

A peculiar feature of the waterwheels is the external bypass on each wheel, which opens as the turbine gates close, due to any reduction of the load on the generators, and which, thereby, relieves the sudden pressure in the supply pipe, and at the same time maintains a constant rate of flow of water into the plant. The rate of flow does not vary more than 3 per cent., and when tripping the circuit-breakers at full load the variation in speed of the wheels is but 12 per cent.

The original installation for power and lighting purposes for this plant consisted of three 150-kw. generators driven by DeLaval steam turbines. This power plant is located at the River Pumping Station of the Cincinnati water-works, which is about ⅓ mile distant from the filter plant. They now form reserve units which may be used when extra power is needed, as for example, when cleaning settling reservoirs or coagulating basins. The waterwheels, however, are entirely sufficient to supply power for the routine operation of the plant, and for lighting and some minor power requirements at the River Pumping Station.

Pumping Machinery.—The pumping station which supplies the untreated water to a filter plant can not, as a rule, be regarded as a part of the filter-plant installation, although usually located in close proximity to the latter. Unless storage reservoirs of large size intervene between the pumps and the filter plant, the operation of the two plants must be coördinated if proper purification of the water is to be effected. For this reason the rate of pumping must usually follow the rate of filtration within comparatively narrow limits.

Types of Pumps.—Both displacement and centrifugal pumps are used for this service, and points of particular advantage may be found in each type of pump according to local conditions. Mr. R. L. Daugherty, in his book on "Centrifugal Pumps," discusses in an impartial manner the relative merits of the two types of pumps. The following statements have been freely drawn from his summaries of the advantages and disadvantages involved in the use of the two types of pumps.

Mr. Daugherty states that in general it may be said that dis-

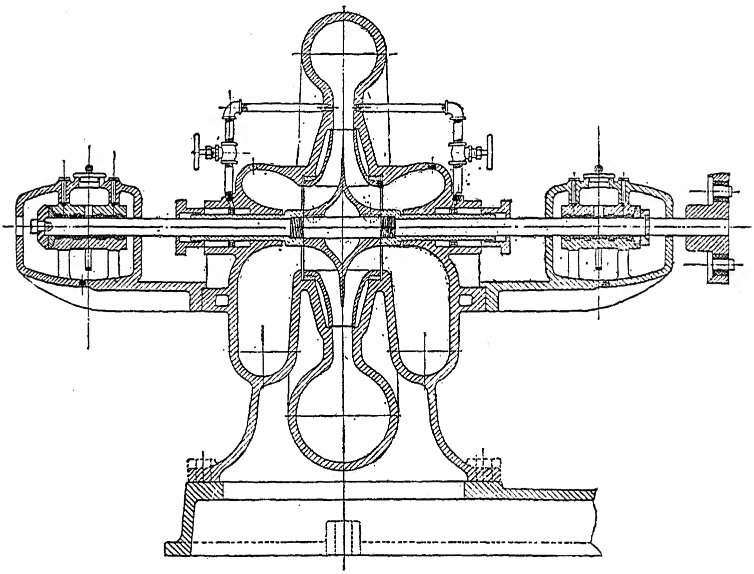

FIG. 110.—Double-suction single-stage volute centrifugal pump.

placement pumps have the advantage where the head pumped against is high, and the capacity small. They may be under some conditions much more economical of power. They are able to lift water from below when starting without being primed. Where either the head or the discharge varies within wide limits, and where they do not maintain definite relations with each other, the displacement pump is perhaps easier and more economical to operate.

Centrifugal pumps may be broadly divided into two classes, viz., turbine pumps and volute pumps (Fig. 110) without diffusion vanes. The latter type of pump is more commonly employed than the former in filter-plant installations. Centriuf-

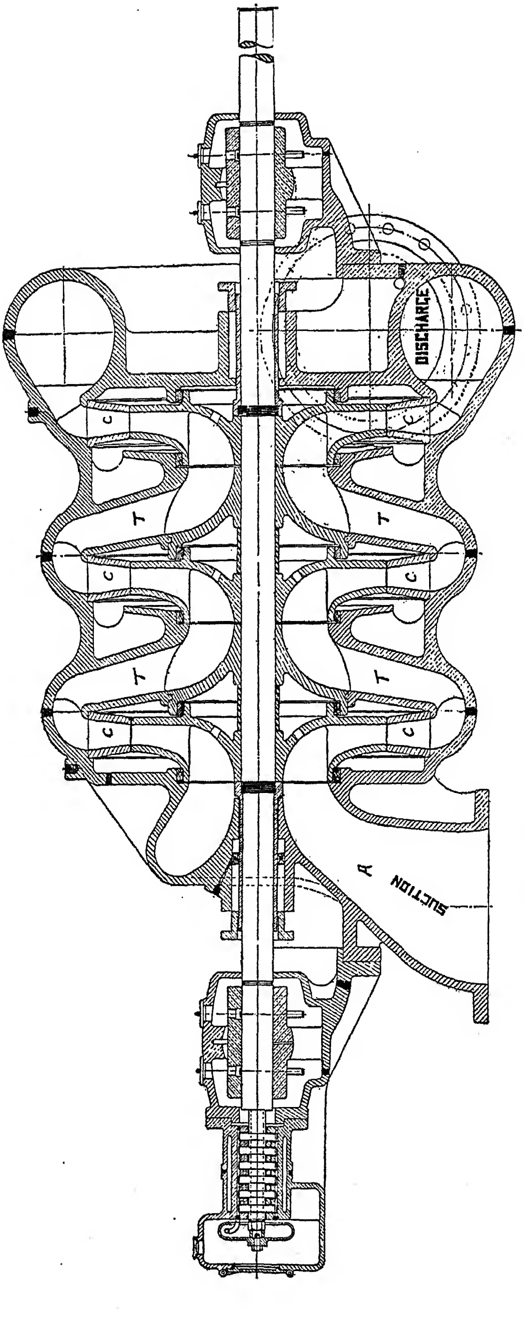

Fig. 110a.—Three-stage horizontal turbine pump.

gal pumps are simpler in construction and generally much less difficult to operate than displacement pumps. A centrifugal pump can handle effectively water containing more or less sand. They are much more economical of space, lighter in weight, and less in cost than reciprocating pumps. They give a continuous and smooth discharge without the pulsations produced by a reciprocating pump. On the other hand, they are usually operated at high speeds, requiring them to be motor-driven, or else direct-connected to a high-speed prime mover like the steam turbine. The motor-driven pump lends itself to a great variety of conditions, and in many places furnishes the most practicable and economical pumping unit, even where overall efficiencies are considered.

With the exception of large pumping engines, which consist of slow-speed steam engines directly connected to displacement pumps, and which give a very high economy of steam consumption, the smaller reciprocating pumping engines are usually less efficient than the direct-connected centrifugal pump. The volume of water discharged by a centrifugal pump varies with the speed of rotation, and is in direct proportion to the square root of the head. The displacement pump also varies its discharge with the speed, but the speed has no relation to the head against which the pump is discharging. For a centrifugal pump the rated head and discharge for the pump will be those values for which the efficiency is a maximum. This value of the discharge is often designated as the *normal* discharge. These values differ for different speeds.

Wash-water Pumps.—Unless it so happens that a pumping station, handling the filtered water from the filter plant, is located near the latter and is able to supply wash water under pressure, it is necessary to install pumps for this service. Since filtered water for filter-washing purposes is usually required in considerable volume at varying intervals, and since this water when required must be delivered in a comparatively short space of time, it is advisable to provide a small reservoir or tank for storing water for this purpose. The tank enables the pumps to be smaller and to be operated somewhat more economically for the reason that they may be run more nearly continuously.

The motor-driven centrifugal pump is well adapted for wash-water purposes. Wash-water pumps usually obtain their filtered-water supply directly from the filtered-water reservoir, or from

a pump well or supply pipe connected with this reservoir. If the pumps are required to lift the water, the suction pipes must be provided with foot valves so that priming of the pumps is not required each time they are started. It has been the author's experience that a clack or flap foot valve is more productive of shock in the pipe line when the pump stops and is more liable to leak, than a number of small circular rubber pump valves operated by springs and set in a cage. The area of the foot-valve opening should be about twice that of the suction pipe. It is usually desirable to place a strainer at the end of the suction pipe in order to protect the foot valve. The openings of the foot valves should be at least 3 ft. under the water surface to prevent vortex action.

The suction lift should not exceed 25 ft. at sea level, and it is always best to keep it as low as possible. The greater the suction lift the greater the liability of air dissolved in the water being liberated. This air may collect at some point in the pipe line, and, as it is always richer in oxygen than the atmosphere, it more actively corrodes the iron with which it comes into contact. According to Mr. R. L. Daugherty, 90 per cent. of centrifugal-pump troubles will be found on the suction side of the pump. The remainder of the difficulties are largely due to end thrust.

The capacity of wash-water pumps, when they are used to supply a tank, depends largely on the capacity of the latter and on the size of the filter plant, viz., on the maximum demand which the filters may make for wash water at any time. The pumping capacity should not be less than 10 per cent. of the filtered water output of the plant, and preferably 12 to 15 per cent. The pressures against which wash-water pumps operate vary with the local conditions. When utilized for filter-washing purposes only, the total head really needed probably never exceeds 50 ft., although other considerations may make it desirable for them to be capable of pumping against a much higher head.

Types of Centrifugal Wash-water and Flushing Pumps Commonly Used.—The kind of centrifugal pump used for wash-water purposes is generally of the volute type as this lends itself better to a varying range of head and to low lifts. The horizontal split case affords easy access to the pump impeller, and the interior of the casing, and makes the removal of the shaft and impeller when necessary, less difficult than in the side-plate casing type.

For single-stage pumps the double-suction inclosed impeller type offers the advantage of eliminating end thrust and considerable wear and friction on bearings, beside permitting a larger volume of water to be handled with a given diameter of impeller.

The advance in late years in centrifugal-pump design enables builders to supply single-stage pumps for considerable higher heads than was formerly the case. From 100 to 200 ft. head per stage appear to be the usual limits within which design is being kept. Multistage pumps, therefore, are not usually necessary in filter-plant operation, where high pressures are not commonly required. The exception to this may be in the case of centrifugal pumps used for flushing out sediment in coagulation and settling basins, where pressures at the nozzles of hose lines should be from 60 to 75 lb. per square inch, and where a pressure of 100 to 120 lb. per square inch may be needed at the pump.

Centrifugal pump casings are usually of cast iron, and the impellers of hard bronze. The portion of the shaft in contact with the water is generally protected with bronze sleeves. Labyrinth rings used to prevent leakage of water from the discharge to the suction chambers are also made of bronze. Ring-oiled bearings are necessary for the high speeds at which centrifugal pumps are usually operated.

Pressure Pumps.—In case hydraulic valves are employed, it is necessary to provide water under considerable pressure to operate them. Other requirements for water under high pressure may exist, such as for the operation of sample pumps, rate controllers, boiler feed lines, sand ejectors, hose lines and so forth. The quantity of water required is not usually large.

For this service an ordinary steam-driven displacement pump may be used, or a motor-driven triplex, single-acting, outside-packed plunger pump. Pumps of the latter type are quite commonly driven through gears, but may be belt- or chain-driven as well. The speed of triplex pumps is from 36 to 50 revolutions per minute. For filter-plant operations pressures in excess of 100 lb. per square inch are rarely required.

Motors for Centrifugal Pumps.—In Vol. 1 of their books on "American Sewerage Practice," Messrs. Metcalf and Eddy have given an excellent summary of the relative merits of different types of electric motors for pumping work. The following statements are in part quoted from this work.

Electric motors for centrifugal pumps should not be too small,

as they may become overloaded under certain conditions. Their capacity should be such as to meet the maximum conditions imposed by the pump. If the head is variable, speed regulation of the motor is desirable. This is possible where direct current is available. For pumping into elevated tanks or reservoirs in most filter-plant installations, the variation in head is usually not great enough to require a variable-speed motor. Where the capacity of the pump is to be varied, and a standard induction motor, which runs at constant speed, is used, the discharge from the pump must be throttled. The shunt-wound direct-current motor is usually employed for driving centrifugal pumps, but a compound-wound motor is better adapted to those cases where the voltage or load fluctuates considerably. Automatically started motors should be compound-wound. The squirrel-cage type of alternating-current motor is also frequently used for driving centrifugal pumps, but it requires a high starting current; the slip-ring motor, which takes a very small excess current at starting, is better in this respect and is to be preferred.

Air Compressors.—Where air is used for agitating the sand bed prior to or at the time of applying wash water during filter washing, a supply of compressed air is necessary. This is usually obtained from rotary blowers of the positive type. These blowers consist of two cam-shaped impellers, which, in rotating inside of a casing, meet in a line of rolling contact, and force the air out from between them. The impellers are operated through gears at a speed of about 200 revolutions per minute, and are generally motor-driven.

It is usually required that the air be discharged at a pressure of 4 lb. per square inch. The capacity of these blowers should be approximately 4 cu. ft. of free air per minute for each square foot of filter surface that may be subjected to the washing process at any one time.

Air Tanks.—A somewhat unique method of providing a storage for compressed air and wash water has been employed at several places in the United States and Canada. It consists in providing a telescopic tank built like a gas holder to contain both air and wash water.

A tank of this character was built in connection with the rapid sand filtration plant at Trenton, N. J., and is described in the *Engineering Record* in the issue of May 9, 1914.

"The tank is of steel and is 40 ft. in diameter. It provides water under a head of 24 ft., and air under a pressure of about 4 lb. per square inch. The floating air chamber is loaded with 300 tons of concrete to produce the required air pressure. The tank provides a uniform pressure of air, and approximately a uniform pressure of water without the necessity of installing a large wash-water pump and blower, and of maintaining a large amount of power ready for use at irregular intervals. The necessary capacities of wash-water pumps and blower are only about one-tenth of that which would be necessary in case of direct delivery of water and air to the filter beds. The tank will supply water

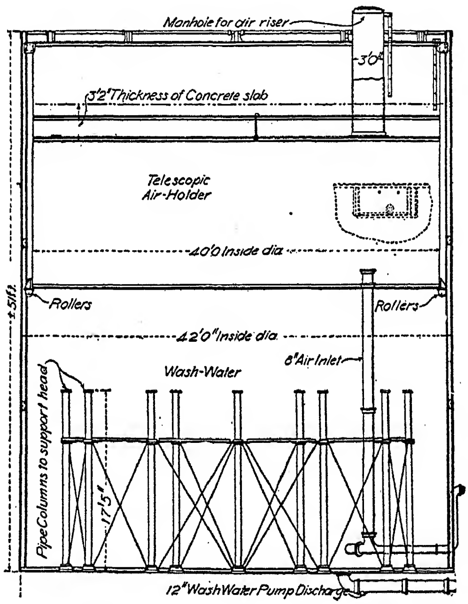

FIG. 111.—Telescopic tank for air and wash water.

and air, without replenishing, for washing two filters in immediate succession; but is so arranged that the pump and blower automatically start to supply water and air to the tank as soon as washing commences. Water is supplied to the filters at the rate of 19 in. vertical rise per minute or 12 gal. per square foot of filter surface."

The accompanying cut of the water and air tank of the telescopic type built for the Montreal Water and Power Co., illustrates the details of this class of storage water and air tank (Fig. 111).

At Columbus, Ohio, the water-purification plant is provided

with three storage tanks for air, which are kept under a pressure of 60 lb. per square inch, and from which the air is drawn through a pressure-reducing valve at a pressure of 2 to 4 lb. per square inch. The air compressor has a capacity of 100 cu. ft. of free air per minute, and is driven by a 15-hp. (220-volt) electric motor. The tanks are each 6 ft. in diameter and 30 ft. long, and are made from ¼-in. riveted-steel plate. They are of sufficient capacity to permit one filter to be washed about every 1.75 hr. for about 3 min. at a maximum rate of 3 cu. ft. per square foot of filter surface per minute.[1]

Wash-water Tanks.—Since in the washing of rapid sand filters a large volume of water is used in a comparatively short space of time, and at irregular intervals, it is more economical to install a storage tank which may be filled between washes that it is to provide power and pump capacity capable of furnishing the volume of water at the required rate during the washing process. Washing a 1,000,000-gal. filter unit of 360 sq. ft. at the rate of 7.5, 10, 12, or 15 gal. per square foot per minute, equivalent to a vertical rise of the wash water of 12, 16, 19, or 24 in. per minute, requires that the total volume of water to be delivered shall be 2,700, 3,600 ,4,320, or 5,400 gal. per minute, respectively.

The storage-tank capacity should be governed by the size of the filter unit and by the frequency of washing. It is usually desirable to have the tank of sufficient size so that at least two filters may be washed without refilling the tank. The size of the pumps required to fill the tank depends upon the probable frequency of washing. It is apparent that they must be able to refill the tank between washes. The frequency of washing is governed by the character of the water being filtered.

As an illustration of the required wash-water tank capacity for a filter plant having ten 1,000,000-gal. filter units, all of which are being operated, let it be assumed that filters must be washed for 5 min. at a rate of 5,400 gal. per minute, and that each filter is to be washed six times a day. Each washing would require 27,000 gal. of water. This volume of water would be required at intervals of 24 min., or at the rate of 1,125 gal. per minute. The wash-water pumps, therefore, would have to have at least this capacity. If the tank is to hold enough water for two washes, then its capacity must be at least 54,000 gal.

In designing the wash-water tank and pumps for a rapid sand

[1] JOHN H. GREGORY: *Proc.* Am. Soc. C. E., Vol. 36, No. I, January, 1910.

filter plant, it should not be overlooked that the maximum demands of the plant and not the average must be provided for. The above illustration is perhaps an extreme, since the wash water would amount to 16.2 per cent. of the total output of the plant; nevertheless, as previously stated, at least 12 per cent. and possibly 15 per cent. of wash water should not at times be regarded as an impossibility.

Wash-water tanks may be built of steel plates or of concrete. If there is no natural elevation upon which the tank may be located, it will have to be placed upon a tower or other suitable foundation, which will provide the head necessary for washing the filters. Wash-water tanks should be covered over at the top in order to keep out the light and thus prevent microscopic plant growths in the water. They should be protected from frost in cold climates where the formation of ice might cause trouble.

Steam Boiler Plant.—The heating of portions of the buildings of a rapid sand filter plant, the necessity for providing steam in the preparation of certain chemical solutions, and the need for a small amount of steam in the laboratories of the plant, make the installation of a steam boiler plant necessary even if not required for power purposes. The exception to this would be, of course, where steam could be conveniently and economically supplied for the above purposes by the boilers of a nearby power plant.

The offices, laboratories, chemical house, and to a certain extent the filter house should all be heated during the colder months of the year. In tropical climates such a procedure is, of course, unnecessary. Where men are constantly at work in the buildings, the temperature should, if possible, be kept sufficiently high so that they are comfortable. The heating of the filter house, the pipe galleries and other parts of the plant in which men are not constantly at work, need only be kept at a moderate temperature to insure against freezing of water in the smaller pipe lines, valves and tanks.

In preparing lime solutions the heating of the water used to slake the lime is practised in a number of plants. In such cases steam under a moderate pressure is required in order to raise the temperature of the slaking water to 150° or 160°F. Closed heaters may be employed for this purpose.

For steam baths, autoclaves, drying ovens and for preparing distilled water, a supply of steam under moderate pressure is

always of much convenience in the laboratories, although other methods of caring for these requirements may be provided.

Since the demand for steam in a rapid sand filter plant, exclusive of that which might be needed for power purposes, is rather small even in the largest plants, the boiler plant can be of moderate size. It should be, however, of ample capacity, so that it may be able to meet the maximum demands which may be made upon it. The steam pressures required will probably be from 60 to 75 lb. per square inch. The quantity of steam needed will, of course, vary for each plant.

Where a fire-tube boiler[1] is used, it is customary to allow 10 to 12 sq. ft. of heating surface per boiler horsepower, or, if 34.5 lb. of water are evaporated per horsepower, then 1 sq. ft. of heating surface will give an evaporation of about 3 lb. of water from and at 212°F. The heating surface in a fire-tube boiler is estimated as about two-thirds of the shell and tube sheets and the external surface of all of the tubes.

The grate area needed must be proportioned to the kind of fuel used and to the draft. With ordinary chimney draft, the grate area should be ⅓ sq. ft. per horsepower, or more. If forced draft is used this area may be diminished. Ordinarily the rate of combustion is 10 to 15 lb. of coal per hour per square foot of grate for anthracite coal, and 15 to 20 lb. for bituminous coal. With forced draft these amounts of coal can be much exceeded.

As it is always desirable to reduce the smoke from a boiler plant to a minimum, the use of mechanical stokers may be advisable for this purpose, although strictly from the point of view of economy their installation in connection with boilers for a filter plant can hardly be considered necessary. The cost of upkeep for most mechanical stokers more than offsets the saving in fuel produced by more uniform firing.

Since a steam-heating plant may be so arranged that a great deal of the condensed steam can be returned to the boiler feed pumps and again used in the boiler, little trouble from scale and incrustation derived from a hard water, which the filter plant may be obliged to purify, need be experienced. In those cases where little fresh water is used, the condensed steam may act corrosively on the iron with which it comes into contact, and should be guarded against by emptying the boiler at times of cleaning and refilling with fresh water.

[1] "American Civil Engineers' Pocket Book," p. 1300.

Refrigerating Plant.—It is not uncommon that water-purification plants are so located that it becomes difficult to obtain ice, which is needed for cooling drinking water, and for ice chests and incubators in the laboratories. If sufficient power is available, it is possible to install a small ice-machine plant, which will not only supply ice, but can be also utilized to cool large refrigerators by means of pipes through which cold brine is kept circulating.

Such a plant has been found a useful adjunct of the Cincinnati water-purification plant, although its refrigerating capacity is equal to only about 2 tons per day. The plant consists of a small vertical ammonia compressor driven by a 5-hp. variable-speed (180 to 250 r.p.m.) motor. The compressor, oil separator, condenser, liquid ammonia receiver, brine cooling system and brine pump occupy but little space in a small room in the basement of the head house. The brine tank, in which are suspended the cans containing the water to be frozen, requires somewhat more floor space, and is located on the first floor of the head house. The cold brine is circulated by a small centrifugal pump through the tank containing the ice cans, and also through a system of piping in a large refrigerator located in the laboratories. These pipes, through which the cold brine is kept circulating, assist in keeping the refrigerator cold, and also in preventing the melting of the ice which is stored in the refrigerator after it has been taken from the cans in which it has been made.

Two drinking fountains in the head house are also arranged so that the water passing to them is cooled by cold brine circulating through a pipe system in a tank of water in which the supply pipe to the fountains is coiled. All of the ice made is "raw water ice," that is it is filtered water frozen in 25-lb. cakes in the small ice cans hung in the brine tank previously described.

Gas for Laboratories.—Laboratories can scarcely be called well-equipped unless gas is provided at various points along the work benches. Where natural or artificial gas is not available from a public supply, which is the condition usually encountered in filter-plant installations, gasoline gas supplied by a gas machine for this purpose should be provided. A proper supply tank for the liquid gasoline, consisting of several shallow pans placed one above the other within a closed tank, is usually buried in the ground outside of the building. An air pump, which is operated either by a weight or by a water motor, forces air into the gasoline tank, where it becomes enriched with the vapors

from the gasoline. Suitable arrangement of pipes from the air pump to the tank and from the tank into the building, furnishes the means for forcing the air into the gasoline tank and for conveying the gas thus formed to the distribution piping in the laboratories. The tank should have a capacity of 100 to 500 gal. of gasoline, depending on the volume of gas likely to be needed. A highly volatile gasoline must be used in the tank if satisfactory results are desired.

The author has found it convenient to provide a double pipe system in the laboratories, one conveying the gas, and the other air piped directly from the air pump. By joining these pipes together at each burner outlet, it becomes possible to regulate the supply of air for each burner, and thereby obtain a proper mixture of gas and air. When the tank is freshly filled with gasoline the gas formed is likely to be too rich and to produce a smoky flame. By adding air at the burner this condition may be avoided and a better and more economical utilization of the gas may be obtained.

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CHAPTER XXIV

THE COST OF CONSTRUCTING RAPID SAND FILTERS

The cost of constructing a rapid sand filter plant is obviously largely dependent upon local conditions. Statements of cost can be only general in character, unless they are actual records of the cost of some plant that has been installed. It is not always easy to itemize the cost of a plant on account of the way in which contracts for the work may be grouped, and also because of inadequate methods of cost keeping. The cost of coagulating basins was touched upon in a preceding chapter, as was also the cost of settling reservoirs. The cost of the filters and filter-plant equipment vary considerably and any attempt to generalize is more or less unsatisfactory.

Mr. G. A. Johnson in a paper entitled "The Purification of Public Water Supplies" (U. S. Geological Survey, Water Supply Paper No. 315) has listed the cost of a number of rapid sand filter plants, describing briefly in each case the salient construction features of the plant, and the bearing which these structural features had upon the actual cost of the plant. Seven of these plants which he describes are listed below.

Approximate Cost of Rapid Sand Filter Plants

Place	Capacity of plant, million gallons daily	Cost per million gallons of filtering capacity
Little Falls, N. J.	32	$15,300
New Milford, N. J.	24	11,000
Harrisburg, Pa.	16	10,300
Binghamton, N. Y.	8	10,800
Watertown, N. Y.	8	11,250
Lorain, Ohio	6	14,2C0
Scranton, Pa	6	13,330

It will be noted that the approximate cost per million gallons of daily capacity is about $12,600. The two plants not listed are those at Cincinnati, Ohio, and Columbus, Ohio. At Cincinnati two large settling reservoirs constitute a part of the purifica-

tion system, although they are utilized also for storage purposes. At Columbus the purification plant is more properly characterized as a water-softening plant, and in any consideration of cost figures this must be constantly borne in mind.

In discussing a paper[1] by Mr. John H. Gregory on the "Improved Water and Sewerage Works of Columbus, Ohio," the author compared the costs of the Cincinnati and Columbus filter plants. These remarks[2] are quoted below for the purpose of showing the difficulty of drawing comparisons where the structures constituting the plants are not used for exactly similar purposes.

"It is obvious that no strict comparison of the costs of similar parts of these two plants is justifiable, since each was designed to meet local conditions. At Cincinnati the plant was required to clarify and purify a turbid water; at Columbus the same conditions were to be met, but, in addition, the water was to be softened. Apparently, the latter function was considered as governing the design of the works at Columbus."

Table 11 in Mr. Gregory's original paper details the cost of the Columbus plant as follows:

Columbus Water-purification Plant—Capacity 30 Million Gallons in 24 Hr.

	Total cost	Cost per million gallons daily capacity
Settling basins	$168,770	$5,630
Head house	39,660	1,320
Air-wash equipment	3,470	120
Lime saturator house	32,550	1,080
Mixing tanks	44,230	1,470
Storage house	12,880	430
Office and laboratory	15,280	510
Filter gallery	102,710	3,420
Filtered-water reservoirs	98,300	3,280
Wash-water tank, pipe and shelter	13,150	440
Supplies for preliminary operation	460	20
Expenses unclassified	1,020	30
Total	$532,480	$17,750

[1] *Proc.* Am. Soc. C. E., January, 1910.
[2] *Proc.* Am. Soc. C. E., March, 1910.

In the author's discussion of Mr. Gregory's figures the detailed costs of the Cincinnati filtration plant were given:

TOTAL AND UNIT COSTS OF MAIN FEATURES OF WORK DONE IN CONSTRUCTION OF WATER-PURIFICATION WORKS AT CINCINNATI, OHIO

	Total cost	Cost per million gallons of daily capacity
Preparation of grounds	$33,359.67	$297.85
Pipe lines between settling reservoirs and head house.	55,354.77	494.24
Head and chemical house. . . .	141,989.85	1,267.77
Coagulation basins, gate houses and pipe lines.	304,913.05	2,722.44
Filters, filter house, piping, sand and gravel.	592,112.30	5,286.71
Piping, valves and gate house between filters and clear-water reservoir.	29,701.91	265.20
Clear-water reservoir	121,362.29	1,083.59
Total	$1,278,793.94	$11,417.80

"At Columbus, the unit costs per million gallons of capacity in 24 hr. appear to be considerably greater for the settling basins and mixing tanks combined, than for the coagulation basins at Cincinnati. The figures for the Columbus tanks and basins are $7,100 per million gallons of capacity, as compared with $2,722 at Cincinnati. In a general way, these parts of the two plants correspond; but it should not be forgotten that at Columbus more elaborate baffling of tanks and basins, more divisions of the flow of the raw and treated waters, and more places for the primary and secondary applications of chemical solutions were needed and provided for, than were required at Cincinnati. The greater combined unit costs of the head house, lime-saturator house, storage house, wash-water tank, offices and laboratories at Columbus, than for the corresponding head house, chemical house, wash-water tank, offices and laboratories at Cincinnati, are similarly explained by the necessity for designing a plant for softening, as well as for clarifying and purifying the water. The combined unit costs for the items noted above for the Columbus plant amount to $3,784, as compared with $1,268 for the Cincinnati plant.

The filters and piping in the Cincinnati plant cost more per million gallons of capacity than did those at Columbus. The figures for Columbus, which include the air-washing equipment, are $3,540, as compared with $5,287 for Cincinnati. However, the filtered-water reservoir at Columbus cost more than that at Cincinnati. The figures

for the Columbus plant are $3,280 per million gallons, and for the Cincinnati plant, $1,084. At the latter plant, the clear-water reservoir is a separate uncovered reservoir, while at Columbus it is directly under the filter tanks, which latter form a protecting roof. Virtually, no great difference in costs exists, if the cost of the filters, piping, and clear-water reservoir of each plant be combined and then compared.

The cost per million gallons of capacity for the whole purification plant at Columbus is stated to be $17,750, which amount does not include engineering; the corresponding figures for the Cincinnati plant as shown above, is $11,418, and this also excludes the cost of engineering. The difference of more than $6,000 per million gallons of capacity is doubtless due to the additional requirement demanded by the local conditions at Columbus, that is, for the softening of a very hard water, and one which is at times subject to rapid fluctuations in its physical characteristics."

In a more recent paper, entitled "Present Day Filtration Practice," read at the annual convention of the American Water-works Association in 1914, Mr. G. A. Johnson discusses the relative costs of slow and rapid sand filters. He concludes that the average cost per million gallons of daily capacity is $32,600 for slow sand filters and $12,100 for rapid sand filters. To these conclusions Mr. John H. Gregory took exceptions in his discussion of the paper.[1] He cites the cost of several rapid sand filter plants to show that Mr. Johnson's average cost of $12,100 per million gallons of daily capacity is much too low. The Columbus, Ohio, plant considered as a rapid sand filter plant cost nearer $15,000 than $13,000 per million gallons of daily capacity as given by Mr. Johnson. Mr. Gregory concludes that the Toledo, Ohio, filter plant,[2] built for a capacity of 60,000,000 gal. per day, cost about $14,500 per million gallons. The 20,000,000-gal. plant at Grand Rapids, Mich., cost approximately $16,300 per million gallons of daily capacity; and the New Orleans, La., 40,000,000-gal. plant cost about $30,200 per million gallons of daily capacity. He estimated that the Jerome Park filter plant, which was to be constructed for purifying the Croton water supply of New York City, and which was to have had a capacity of 320,000,000 gal. per day, would have cost approximately $20,000 per million gallons of daily capacity.

[1] Reader is referred to original paper for discussion of slow sand filter costs.

[2] *Eng. Record,* Nov. 26, 1910.

Referring to the omission by Mr. Johnson of the cost of the settling reservoirs ($1,521,000) from the total cost of the Cincinnati filter plant, and assuming that it might be permissible to charge at least half of the cost of these reservoirs to the purification plant, Mr. Gregory concludes, that the cost of the latter would be about $18,200 per million gallons of daily capacity. He sums up his conclusions by stating that the weighted average of the unit costs of the Columbus, Grand Rapids, New Orleans, Toledo and Cincinnati plants is $18,600 per million gallons of daily capacity, which is over 50 per cent. higher than the weighted average cost of $12,100 stated by Mr. Johnson.

It should be noted that the first three of the five plants named above were designed not only as filter plants but as water-softening plants as well, and that rather exceptional local conditions involving special engineering structures probably account for the greater costs at each of those plants where the unit costs are high.

The original purification works of the City of St. Louis, Mo., were composed of a number of basins, which were utilized for plain sedimentation purposes. Later, these basins were remodeled so that they might act as coagulation basins following the introduction of chemical coagulants. Additions to this plant have been made from time to time, the most recent being a filter plant with its appurtenances, and having a daily capacity of 160,000,000 gal. The cost of the filters, buildings, equipment, and such modifications of the existing plant as were needed, was $1,357,289.15.

According to the annual report of the St. Louis Water Department ending April 1, 1915, "the total cost of all parts of the purification plant as now operated is estimated at $3,549,000 or $22,200 per million gallons rated capacity. This relatively high unit cost is explained by the fact that the capacity of the basins already in place is much greater than would have been necessary for the filter plant only, and the basin cost is, therefore, disproportionately high."

A somewhat detailed table of costs for the 30,000,000-gal. filter plant at Trenton, N. J., is given below. The figures are based on the prices bid for the work (see *Engineering Record*, May 9, 1914, p. 543).

Estimated Cost of Construction of Trenton Filter Plant

Earth excavation, 19,000 cu. yd	$16,530
Rock excavation, 75 cu. yd	113
Cement, 14,000 bbl	18,070
Steel reinforcement, 650 tons	45,500
Reinforced concrete, 8,000 cu. yd	53,600
Concrete not reinforced, 1,500 cu. yd	10,050
Cast-iron bell and spigot pipe, 450 tons	15,840
Cast-iron bell and spigot specials, 42 tons	3,024
Valve chamber	231
Two 30-in. gate valves	850
Screens and hoists	2,292
All buildings	48,600
Heating system	3,200
Low-lift pumping equipment	45,225
Filter equipment and piping	74,500
Strainer system	33,000
Filter gravel, 440 cu. yd	1,778
Filter sand, 1,100 cu. yd	4,444
Wash-water and air tank	9,503
Engineering and miscellaneous	20,000
Total estimated cost exclusive of land	$406,350
Cost per million gallons of daily capacity	$13,545

The estimated cost of the Baltimore filtration plant, as made by Mr. George W. Fuller in his preliminary report as consulting engineer, is as follows:

Estimated Cost of Construction of Baltimore Filter Plant, Daily Capacity 128 Million Gallons

Coagulating and settling basins and appurtenances, including land damages	$370,000
Filters and appurtenances, including land damages	900,000
New covered filtered-water reservoirs	150,000
	$1,420,000
Add 15 per cent. for engineering and contingencies	213,000
Total	$1,633,000
Cost per million gallons of daily capacity	$12,760

From the statements made and the figures given in the preceding pages, it is apparent that no exact estimate of the average cost of rapid sand filter plants can be made, and that each case must be considered by itself. The special modifications of the

design of the plant required to properly purify the kind of water that must be treated, the topography of the site on which the plant is to be built, the proportion of the cost of storage reservoirs and of pipe lines and tunnels conveying water to and from the purification plant proper and which should be charged to the cost of the latter, and the cost of the land, are all elements entering into the computation, and concerning which engineers may not in some cases agree. The cost figures should, therefore, be considered as approximte only.

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CHAPTER XXV

RATES OF FILTRATION, LOSS OF HEAD AND WASHING OF RAPID SAND FILTERS

There are three important features of the operation of rapid sand filters that require special consideration. These are the rate at which the filter is operated, the loss of head produced by the flow of water through the filter, and the method used in washing the filter bed.

RATES OF FILTRATION

The chief distinction between slow sand and mechanical or rapid sand filters is the rate at which the water passes through the sand bed. The average rate of flow at which slow sand filters are now operated in the United States is considerably greater than formerly. This increase in the rate has been made possible in many instances by the employment of methods of preliminary treatment, consisting of sedimentation produced with the aid of coagulants, and by "roughing filters." In the United States a rate of 6,000,000 gal. per acre daily is not usually exceeded, and the average is about 4,000,000 gal. per acre per day.

In the operation of rapid sand filters, however, the rate does not differ materially from that first established, namely, 125,000,000 gal. per acre daily. Higher rates than this are sometimes employed, but unless conditions are unusually favorable, are not often used. Mr. Nicholas S. Hill, Jr., in a paper on "Modern Filter Practice" read before a convention of the American Water-works Association in 1913, gives a table of statistics relating to quite a number of filter plants in the United States, and which, among other data, includes the rates of filtration employed.

Mr. Hill, in commenting on this data, states in substance that with a few exceptions, such as at Elmira, N. Y., Hackensack, N. J., and Harrisburg, Pa., where the rates are 150,000,000, 180,000,000, and 166,000,000 gal. per acre daily, respectively, it will be noted that the rate of filtration is almost always 125,000,000 gal. per acre per day. The size of the filter units, and consequently their capacity, has been increasing during the past 15

MECHANICAL FILTERS

Location	Date	Plant capacity (mil. gal.)	No. filter units	Unit size		Unit capacity	Rate per acre	Filter material (inches)		Sand		Per cent. wash water	Rate applied in rise per min.	Air wash
				Dimensions	Area (sq ft.)			Sand	Gravel	Eff. size (mm)	Un. coef			
Elmira, N. Y. . .	1897	7.0	21	13×Dia.	132	0.33	150	30	6.0	0 56	1.43	2.8	14.0	no
Little Falls, N J. .	1902	32.0	32	24×15	360	1.00	125	30	12.0	0.44	1.47	3.5	12.0	yes
Hackensack, N. J..	1904	48.0	16	46×25	1,000	3.00	180	30	12.0	0.50	1.30	2.7	8.5	yes
Ithaca, N. Y	1904	3.0	6	16×11	176	0.50	131	30	?	0 39	1.50	3 8	?	?
Charleston, S. C...	1904	6.0	12	15×Dia.	176	0.50	100	50	12 0	0 70	1.30	4.0	13.0	no
Harrisburg, Pa.....	1905	12.0	10	27×16	432	1 00	166	30	7 0	0 38	1.30	2.3	9.0	yes
Marietta, Ohio....	1906	2 4	8	20×10	200	0.50	81	36	?	?	?	6.5	11.0	no
Cincinnati, Ohio...	1908	112 0	28	28×50	1,400	4 00	125	30	7.5	0 32	1.20	3 0	24.0	no
Columbus, Ohio...	1908	30 0	10	46×26	1,089	3.00	125	30	10.0	0 41	1 36	2.5	12.0	no
Bristol, R. I.	1908	3 0	6	15×12	180	0 50	125	30	6 0	0.45	1.74	7 0	14.0	yes
Winfield, Kan	1908	2 0	4	18×10	180	0.50	121	48	?	?	?	4.0	?	no
New Orleans, La...	1909	60.0	10	53×27	1,431	4.00	101	36	8.0	0.35	1.65	1.6	24.0	no
Louisville, Ky......	1909	36 0	6	72×30	2,160	6.00	125	24	10.0	0.36	1.50	2 0	22 0	no
Sandusky, Ohio.....	1909	6 0	6	20×18	360	1.00	100	36	10.0	0.40	1 30	3 5	21 0	yes
Toledo, Ohio	1910	34 0	34	22×16	352	1.00	125	30	9 0	0.40	1.45	2.1	14.0	yes
Wilkinsburg, Pa	1910	12 5	10	22×20	450	1.25	120	36	8.0	0.40	1.60	2.3	13.0	yes
Newport, R. I... .	1910	7 5	6	24×14.5	348	1.25	156	27	18.0	0 40	1.70	3.5	16.5	yes
Kansas City, Kan..	1911	12 0	10	32×20	480	1.25	125	30	?	?	?	?	30.0	no
Cohoes, N. Y......	1911	10 0	10	23×15	347	1.00	100	24	12.0	0.40	1.65	4.0	?	?
Bangor, Me	1911	8.0	8	24×18	432	1.00	100	27	9.0	0.59	1.25	4.5	12.0	yes
Fort Worth, Tex....	1911	5.0	4	25×20	540	1.25	125	34	11.0	0.39	1.42	?	14.0	yes
Ottumwa, Iowa.....	1911	4 0	4	24×14.6	348	1.00	125	27	9.0	0.50	1 40	3.5	12 0	yes
Minneapolis, Minn	1912	40 0	12	51×23	1,173	3.25	125	30	8.0	0.39	1.65	0.5	24.0	no
Grand Rapids, Mich..	1912–13	20 0	10	36×23	738	2.00	118	30	8.0	0.39	1.60	4 6	24.0	no
Niagara Falls, N.Y.	1912	16 0	16	25×14.5	362	1.00	125	30	9.0	0.35	1.60	2.5	10.0	yes
Evansville, Ind....	1912	12.0	12	23×16	368	1.00	125	30	9.0	0 26	1.83	3.3	14.0	yes
Fargo, N. D.......	1912	6.0	6	21×17	357	1.00	125	30	10.0	0.35	1.40	1 5	11.0	yes
Rock Island, Ill..	1912	6.0	6	21.5×17	365	1.00	125	30	10.0	0.32	1 65	1.2	14.0	yes
Grand Forks, N. D	1912	4 0	8	18×11	198	0.50	110	30	6.0	0.39	2 10	4 0	?	?
Albany, Ore......	1912	3.0	6	22.5×10.5	184	0.50	125	37	9.0	0.36	1.30	?	12.0	yes
Minot, N. D . ..	1912	1.5	3	14.6×12	181	0.50	125	34	30.0	?	?	3.0	13.0	yes
Trenton, N J.. ..	19–	30 0	16	30×24	652	1.87	125	30	10.0	0.39	1.65	?	18.0	yes
Evanston, Ill	19–	12.0	6	36×23	846	2.00	125	30	8.0	0.65	1.65	?	?	yes
New York, N Y....	19–	320.0	80	50×34	1,700	4.00	125	30	8.0	0.65	1.65	?	?	

years. In 1900 the average daily capacity of a filter unit was about 500,000 gal., having a sand area of 175 sq. ft. At the present time units of 4,000,000 gal. (1,400 sq. ft. of sand) daily capacity are used in the largest plants, while the average capacity is probably between 1,000,000 (350 sq. ft.) and 2,000,000 (700 sq. ft.) gal. per day.

Uniformity of Rate of Filtration.—It is of the utmost importance that the rate of filtration shall be uniform over the whole area of the filter bed. Any sudden increase in the rate of flow is likely to cause the bed to break through at some point, and thereby seriously impair the quality of the effluent. A sudden decrease in the rate of flow is of much less consequence, so far as it may affect the quality of the effluent, unless air, which may have come out of solution in the water or have entered the filter through small cracks, should be liberated from the bed in consid-

erable quantities; in which case the disturbance of the surface of the filter bed is likely to result in an imperfectly purified effluent. Slow and steady changes in the rate of flow within certain limits are permissible, and are not infrequently made use of where insufficient storage capacity for the filtered water makes it necessary that the filters should follow closely the rate of consumption. How great a change in the rate of flow can be permitted, without affecting the quality of the effluent, depends

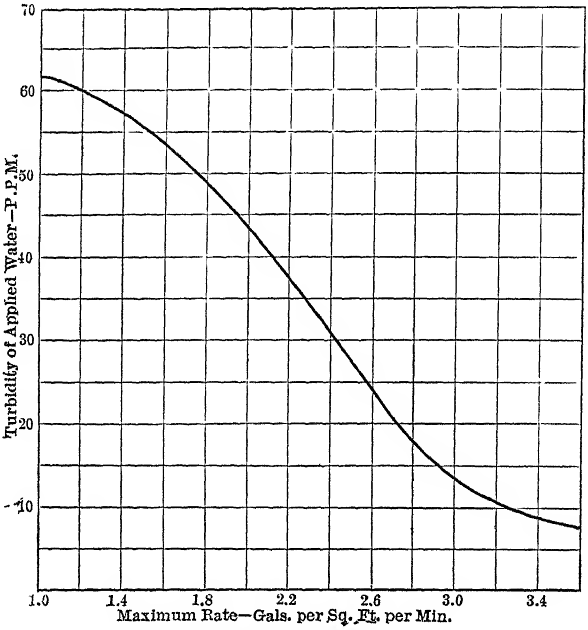

Fig. 112.—Maximum effective rate of filtration for different turbidities of applied water.

upon the condition of the sand bed and the character of the water being filtered. A bed of fine sand upon which there had been deposited a good protective colloid coating, and to which was being applied a well-coagulated water that was not too heavily charged with sediment, could stand a much greater shock produced by a change in the rate, than if the reverse of these conditions existed. Local conditions must be carefully studied before attempting to modify rates of filtration to any marked extent, and especially is this necessary where fluctuations in the rate are to be used continuously in order to follow changes in the rate of consumption of the filtered water.

An interesting illustration of the maximum effective rate of filtration with water of varying turbidity has been furnished the author by Mr. H. W. Streeter, who has plotted the data (Fig. 112) obtained in operating a rapid sand filter plant at Clarksburg, W. Va. It will be noted that rates of filtration varying from 1 to 3.4 gal. per square foot per minute, corresponding to approximately 62,700,000 and 213,000,000 gal. per acre daily, respectively, have been plotted against turbidities in the applied water ranging from about 8 parts to 62 parts per million.

Mr. Streeter's comments are as follows:

"The data for this curve were obtained from about 30 tests of individual filters, and extended over a period of about 1 year. In making a test the filter was started at a low rate, which was slightly increased at intervals of about 20 min., or approximately the time required to displace three times the volume of water passing through the sand. These successive increases in the rate were continued until a rate was reached which produced a turbid effluent. The next lower rate than this (usually 0.2 gal. per square foot per minute) previously tried and found effective under the above conditions was taken as the maximum effective rate for the given applied water turbidity existing at the time of making the test.

"Tests were carried out at various losses of head, and the aim was to distribute them evenly, so as to have observations weighted for each loss of head, that is, to have about the same number of tests of a certain applied water turbidity for each loss of head.

"The curve is well defined for turbidities of applied water below 50 parts per million. Above this there were not so many observations possible, because the applied water turbidity was not allowed to exceed this limit more than a few times. Above 50 parts per million of turbidity there seems to be a reversal of the curve. This means, of course, that the filters could not handle the higher turbidities with the same degree of effectiveness as they did the lower turbidities, even with very low rates of filtration. With very low applied water turbidities, however, disproportionately high rates of filtration were indicated as being possible.

"The curve relates only to the removal of turbidity, and does not attempt to define what the permissible rates of filtration for satisfactory bacterial removals might be under the same conditions. However, satisfactory results from this standpoint were always obtained, when the removal of turbidity was complete, by disinfection of the filtered water."

Rate Controllers.—The maintenance of a uniform, or of a slowly changing, rate of filtration is made possible by automatic

rate controllers. The mechanism of a number of these devices has been previously described, and only a few points of general interest concerning their operation and care need be mentioned.

When first installed, rate controllers should be carefully calibrated for the conditions under which they are to operate. This is easily accomplished by noting the distance the water falls in a filter tank in a given time when the supply to the tank is cut off and when the filtered water is discharging through the controller in the usual manner. By calculating accurately the volume of water filtered from the area of the tank and the observed fall of the water during the measured period of time, it becomes possible to estimate the rate of flow. Such measurements are facilitated by a gage placed in the water above the sand in the filter tank, and on which are a series of hooks placed one above the other at known distances. By using a stop watch the time required for the water level to fall from one hook to the next lower can be accurately gaged.

From time to time the adjustment of rate controllers intended to operate at constant rates should be undertaken, in order to make sure that they are in proper working condition. Corrosion of metal surfaces in moving parts or incrustations or tuberculation of iron piping, or the wearing out of diaphragms or other short-lived parts, are the obvious reasons for periodical adjustments. Where variable rates of flow are desired, fully as much, if not more, care should be given to these machines in order to see that they are controlling properly. Checking the operation of any rate controller of the Venturi type is easily accomplished, if rate-of-flow gages form a part of the equipment.

In those plants which are not provided with rate controllers, or if for reasons of cost they can not be installed in a new plant, the rate of filtration may be in a measure controlled by other methods, and of which advantage should always be taken. A maximum rate may be fixed by some form of orifice plate inserted in the effluent pipe or by skillful handling of the effluent valve itself; by throttling it when the filter is clean and gradually opening it as the filter becomes clogged, it is possible to maintain a fairly uniform discharge throughout the period of service of the filter. Inattention to the rates of filtration in plants not provided with automatic controllers is undoubtedly the cause for the poor quality of the effluent which they so frequently produce.

LOSS OF HEAD

The pressure required for operating a filter varies with the type. Pressure filters are operated under considerably more head than those of the gravity type.[1] As the period of service of a filter lengthens, or in other words as the sand bed becomes more and more clogged with the suspended matter strained out of the water, greater and greater pressures are needed if the rate of flow is to be kept uniform.

A clean filter of the gravity type, when first put in operation, offers but little frictional resistance to the flow of the water, and rates of filtration far in excess of those which would be hygienically safe could be maintained for short periods. To overcome this difficulty resistance to the flow of the water must be introduced. This may be accomplished by throttling the effluent valve of the filter, or better by the use of a rate controller, whose function is to automatically insert this frictional resistance to the flow of the water when the filter is clean, and to withdraw it as the friction head produced by the clogging of the sand bed increases.

Total Head.—The difference between the level of the water on the surface of the filter and the level at which it discharges through a trapped outlet, or if the outlet pipe is submerged, at the level of the water in the tank or reservoir in which submergence is effected, measures the total head under which the filter is operating. For the gravity type of rapid sand filters the total head available is usually from 12 to 15 ft. Variations in the available head are due to changes in the level of the water on the top of the filters and to fluctuations in the level of the water in the receiving basin or reservoir.

Positive and Negative Head.—The total head of water available for operating rapid sand filters of the gravity type is increased by prolonging the discharge pipe with its trapped outlet to a point below the level of the floor of the filter. This is simply a device for increasing the distance between the level of the water in the filter and that at which it discharges. This device has been the subject of patent claims, the validity of which have

[1] Mr. George W. Fuller, in the report on the investigations at Louisville, Ky., states that the Western pressure filter had a minimum available head of 115 ft. The filter was washed when the loss of head reached 50 ft. The sand in this pressure filter was 49.5 in. in depth, and had an effective size of 0.44 mm.

been the cause for much litigation. The water flowing through this so-called "downdraft tube" is claimed to exert "suction" on the filter bed by the production of a partial vacuum within the latter, and to produce a number of secondary effects of a beneficial character. When these conditions exist it has become customary to regard the total head acting in such filters as divided into two parts, namely, "positive head" and "negative head." The significance of these claims will be somewhat better understood by a more complete discussion of the "loss of head" in a filter.

Loss of Head.—The head lost in the passage of water through a filter is the sum of the heads required to produce the rate of

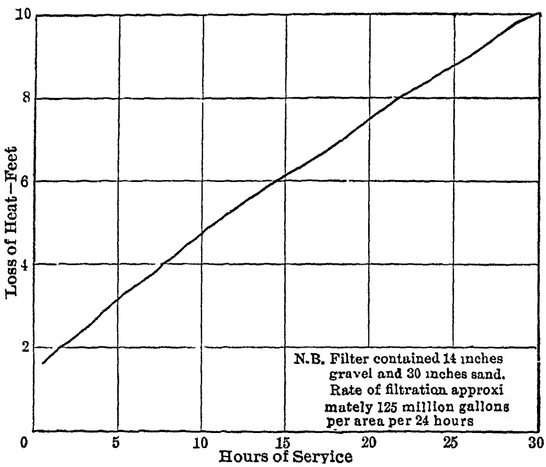

FIG. 113.—Typical loss of head curve for a rapid sand filter.

flow or velocity head, and the head required to overcome all frictional resistance throughout the whole system, or friction head. The velocity head is small as compared with the friction head, which is by far the largest factor, and which is produced principally in the sand layer.

The loss of head increases directly with the rate of flow, with the depth of the sand bed, and with the square of the effective size of the sand grains. The loss of head is greater at lower temperatures than at higher ones, being due to the increased viscosity of the water at the lower temperatures. The decrease in porosity of the filter bed is due to the cumulative effect of suspended matter strained out of the water, and under some circum-

stances to the entrainment of air in the interstices of the sand bed. The effect of the sediment in increasing the loss of head is the factor of the greatest importance, and, naturally, the character and quantity of sediment determine in a large measure the rate at which the head is lost. Trapping of liberated bubbles of air in any quantity in the bed causes a rapid loss of head.

It is obvious that the zone of greatest frictional resistance in a filter bed during filtration is most likely to occur at or near the surface of the sand bed. As the period of operation of the filter lengthens, the resistance becomes greater. A typical loss-of-head curve taken from one of the Cincinnati, Ohio, filters is shown in the accompanying cut.

This curve merely shows the rate at which the available head is used up. When the total head is used up the filter must be taken out of service and cleaned in order to restore it to its original condition. The maximum rate of flow of a filter operated at variable rates must gradually diminish as the filter becomes clogged, since the decrease in the available head necessarily limits the output of the filter even though the controller orifice is wide open.

Partial Vacuum in Filter Bed.—By inserting glass tubes in the side of a rapid sand filter tank of the gravity type, so that the ends of the tubes project into the filter bed at different levels, it will be observed, that as filtration proceeds the water will lower in the tubes and finally disappear at the point where the tube enters the tank. If the tube is trapped, it will be noted that the water may drop below the level at which the tube enters the side of the tank, indicating a pressure less than that of the atmosphere at this point. The distance which the water drops in the trapped tube below the level of the point at which the tube enters the filter bed is a measure of the partial vacuum at this plane in the filter bed.

Mr. George W. Fuller has clearly described the conditions in a rapid sand filter when a partial vacuum exists, in his testimony as an expert witness in the case of the New York Continental Jewell Filtration Co. *vs.* the City of Harrisburg, Pa. (1910).

"A 'partial vacuum' occurs in a filter under those conditions where the pressure becomes less than atmospheric pressure. This is brought about in filters when the 'loss of head' through a section of a filter becomes greater under the given conditions of operation than is the depth of the water measured from the top of the water on the sand

bed down to the plane or bottom of the section of the filter bed in question. A partial vacuum is most easily noted in the case of filters which have a series of glass tubes connected at various points on the side of the filter. If these observation tubes should not be trapped, but have a direct horizontal connection into the side of the filter, a partial vacuum would be found to exist when the water disappears in the glass tube connected to the filter at the point under consideration. So long as the water stands in the observation tubes above the level where connection is made with the filter, no partial vacuum exists. If the observation tubes connected with the filter at various points on the sides are trapped down below the elevation of the connection through the filter wall, then there is an opportunity for measuring the degree of partial vacuum which exists. Thus, if the water should stand in an observation tube 1 ft. below the level where this tube is connected with the sand bed of the filter, then the degree of partial vacuum is stated to be equivalent to a column of water 1 ft. in height.

"For a partial vacuum to be secured in the practical operation of filters, it is necessary for atmospheric pressure to be excluded from within and below the filter, and this is done by preventing air entering the sides of the filter and by having the filter outlet either submerged in a body of water, or else trapped. The purpose of this, of course, is to prevent the entrance of air from below. If this were not done air would enter from below and displace in part the water within a filter below the section where a partial vacuum is developed. Looking at it in the light of the relation to the 'loss of head' in a filter, it may be said that a partial vacuum occurs in a filter with a trapped outlet, when the total loss of head under the given conditions *exceeds* at any plane the depth of water measured from that plane up to the top of the water in the filter above the sand bed. Where the outlet is trapped and no air enters the sides of the filter, losses of head in excess of that just stated are utilized. This is accomplished by virtue of the partial vacuum from the utilization in part of the atmospheric pressure. The extent to which use is made of atmospheric pressure exerting itself upon the surface of the water above the sand bed in the filter for purposes of pushing the water through the filter is proportional to the degree of partial vacuum secured."

Mr. Fuller points out in his testimony following that quoted above, that a

"downdraft tube extending below the filter floor is not necessary for the production of a partial vacuum, and that the 'lower' portion of the filter itself serves as a downdraft tube as regards that portion lying beneath the plane where a partial vacuum first develops. In other words, if a partial vacuum should develop 1 in. beneath the surface of

a sand layer in a filter bed, the lower portion of the filter tank enclosing the portion of the sand bed and other constructions below the plane cited would constitute a downdraft tube."

In defining what is commonly understood as "negative head," Mr. Fuller states that it has become customary to divide the total head into two parts, namely "positive head" and "negative head."

"Scientifically speaking, this is done accurately when the positive head is called that which exists above the uppermost portion of the filter wherein a partial vacuum is produced. A negative head is either obtained by difference, or it may be defined as a head causing the water to move from the top portion of the section containing a partial vacuum to the plane of the delivery pipe which controls the amount of head which can be made available as a total."

The term "negative" applied to any portion of the forces tending to produce flow of water through a filter is confusing and unnecessary. The author is indebted to Mr. C. N. Miller for a clear and concise discussion of the hydraulics of the flow of water through a filter. In this discussion the particular case of the so-called "negative head" is treated from a mathematical standpoint. This matter will be found in an appendix (A).

Washing Rapid Sand Filters.—The cleansing of the filter bed at the end of each period of service is an extremely important part of filter-plant operation. Unless this cleansing process is effective, a gradual deterioration in the condition of the bed must ensue, with the consequent production of a more or less poorly purified effluent.

Probably no part of rapid sand filter-plant operation has received as much study as the process of washing. The mechanical construction of the strainer system, the method of distributing the wash water, and the manner of agitating the sand, all present problems of more or less difficulty. The construction of strainer and agitating systems has been described in a preceding chapter, and only the various methods of washing will be discussed here.

Essentials for Efficient Washing.—Any method of washing, (Fig. 114) to be effective, must remove the sediment which has accumulated during the preceding period of filtration. This sediment may have penetrated into the bed for some distance, or it may be all practically within an inch or two of the surface, depending upon the amount and character of the sediment, the amount of the coagulant employed, the size of the sand particles

and the rate of filtration. The uniformity of the discharge of wash water is, of course, dependent upon the design of the strainer system. The loss of head through the openings through which the wash water enters the filter bed must be as uniform as possible for all sections of the bed. In the modern type of filter the gravel layer, or its equivalent, acts as a second distributor of the upward-flowing wash water, and prevents jet action immediately above the individual strainer openings or wash-water inlets.

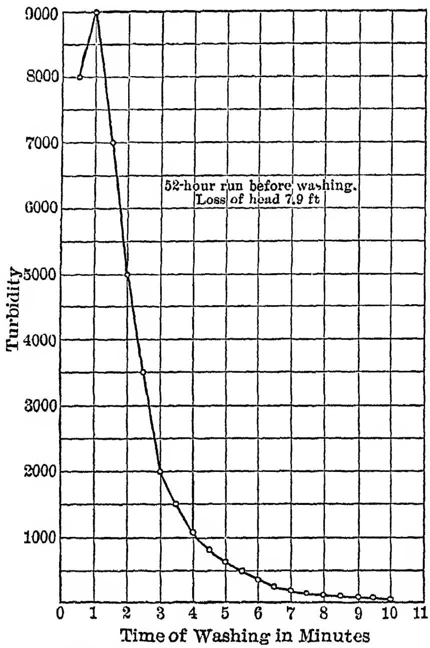

FIG. 114.—Relation between turbidity of wash water and length of time of washing.

Development of the Methods for Washing Rapid Sand Filters.—Since the sand in any properly designed filter bed should retain all of the applied suspended matter, it alone is really in need of cleansing. Consequently methods for agitating the sand bed prior to or during the upward flow of wash water have always formed a part of the washing operation. Three methods of agitation have been developed, one of which is now practically obsolete, while the other two are in active use today.

The early type of rapid sand filter consisted merely of a

cylindrical tank in which the sand bed rested directly upon the strainer system. In these tanks rotating rakes stirred the sand to nearly its full depth as the wash water flowed up through the bed. The bed increased in volume somewhat under this operation, and the agitation produced by the rakes assisted the wash water in removing the particles attached to the sand grains.

In order to prevent the clogging of the strainer openings by sand particles, it became necessary to place between the sand bed and the strainer system a layer of coarser material which was so graded in the size of its particles, that at the bottom they would be too large to pass through the strainer openings, and at the top too small to permit the sand to pass downward. Graded layers of gravel came into use for this purpose, and furnished a satisfactory protection to the strainer system, and a support for the sand bed.

With a gravel bed which could not be disturbed and which supported a sand bed which it was desirable to agitate in some manner in order to make the wash water more effective, it soon became evident that rakes mechanically rotated could not be safely employed. Moreover, the increasing size of filter units, and especially the introduction of rectangular tanks in place of the circular tanks, so increased the mechanical difficulties involved, that designers sought other means for agitating the bed. Except in old filter tanks, which may be still in use, or in quite small plants, where circular tanks are still sometimes installed, this method of agitation is no longer commonly employed.

Velocity of Wash Water in Feet per Minute

Gravity filters	Agitation with rakes		
	Average	Maximum	Minimum
Jewell filter	0.62	1.16	0.19
Warren filter	0.81	1.82	0.39
	No agitation except that produced by rising wash water		
Western filter	1.01	1.26	0.74

Note.—It is stated by Mr. Fuller that the washing of the Western filter, in which no rakes were used, was generally unsatisfactory. The effective size of the sand in these filters ranged from 0.43 to 0.51 mm., and the depths of the beds were from 27 to 31 in.

The rate at which the wash water was applied in this method was relatively low, and, moreover, varied considerably. Mr. George W. Fuller in the report of the experiments with the Jewell, Warren and Western gravity filters at Louisville, Ky., gives some figures on wash-water velocities which have been recalculated to show velocities above the sand line and not in the bed itself. The results are shown in the preceding table.

Compressed Air for Agitating the Sand Bed.—The use of compressed air for agitating the sand bed was advanced as a substitute for mechanical rakes, and soon came into general use. By this method of agitation the air is applied at a low pressure, and in a volume of 2 to 5 cu. ft. per minute per square foot of sand area. The air is usually applied to the tank after the water has been drawn down, so that only a few inches covers the surface of the filter bed. The air is generally applied for several minutes prior to the application of the wash water, and produces in its escape through the water a "boiling action," which gives an observer the impression that the sand is being violently agitated. This is not the case, however, and only a very slight movement of the sand is produced.

The author was enabled some years ago to actually observe the effect of air and water applied to a sand and gravel filter bed in an experimental filter having glass sides. The following is quoted from the published account of these experiments, which appeared in the Report of Mr. George H. Benzenberg, Chief Engineer of the Commissioners of Water-works of Cincinnati, in Appendix C.

"On top of the releveled layer of gravel a 30-in. layer of sand was placed, and the effect of air and water currents observed. Water admitted under a pressure of 5 lb. per square inch had no very marked effect on the gravel. The sand simply became less dense and floated up a little, as has been observed before.

"Air was then forced in through the gravel and sand, the water being shut off. Under a gage pressure of about 3 lb. a volume of air was discharged into the filter equal to 6.86 cu. ft. The agitation of the sand was less marked than the writer expected, and other observers, when first seeing the effect of the air agitation, have noted this fact with considerable surprise. By watching the water above the sand it would be concluded that a very general agitation of the sand was taking place.

"A peculiar effect of the air current was that beside an ascending current moving more or less sand with it, a descending current would often be noticed carrying sand in the opposite direction. This pecu-

liarity is much more marked when air and water are being applied than with air alone. The gravel was more or less disturbed by the rising air currents.

"Forcing in air and water together was next tried. It was immediately apparent that a far better agitation of the sand took place when the two were applied together. The less dense condition of the sand afforded a better play for the air currents, and they moved the sand about much more easily. Ascending and descending currents of sand could easily be traced. The gravel naturally suffered a great deal by the more violent agitation. At the end of the wash the sand and gravel were more or less mixed up, and presented a very irregular outline. It was quite apparent that the gravel layer ought not to be subjected to the violent action of the air current if it was to retain its uniform distribution over the bottom of the filter, unless it was held in place in some manner."

The rate at which wash water is applied to a filter, in which compressed air is used for agitation, does not differ materially from those employed when rakes were used to agitate the bed. The velocity of the rising wash water is usually from 10 to 15 in. per minute, measured in the tank above the sand line. Lower velocities must be used if air and water are applied at the same time, unless ample distance between the normal sand line and the edges of the waste troughs is provided to prevent wastage of sand at the above velocities. The escape of the air from the rising wash water tends to carry the sand much higher than would the water alone. Under these conditions too high velocities may cause loss of sand over the waste troughs as stated above.

The process of washing a filter may take from 10 to 20 min., depending upon the rate of application of the wash water, the amount of water required to clean the bed, the ease with which valves may be manipulated, and air compressors and pumps started and stopped. Hydraulically or electrically operated valves in the larger plants facilitate the operation greatly. The washing process should always be carefully watched in order to note whether the wash water is rising uniformly, whether the waste troughs are flooded, and whether the sediment is being properly carried away, *i.e.*, does not remain suspended between the troughs instead of flowing toward and into them.

High-velocity Method of Washing Rapid Sand Filters.—The slight agitating effect of compressed air upon a filter sand, as shown in the previously described experiments of the author, being regarded as unsatisfactory, further investigations were made in

order to discover, if possible, a method of agitating and washing the sand which more nearly corresponded with the scrubbing effects produced by the rotating mechanical rakes formerly used. As it was noticed that by increasing the velocity of the wash water in its passage through the sand bed, the latter was floated upward to higher levels, and that the sand grains moved more freely, rubbing against each other as they were forced upward by the rising currents of water, it seemed as though it offered another method for agitating the sand and had the added advantage of cleansing the sand grains at the same time.

As a result of the experimental work which necessarily followed these tentative conclusions, there was developed what is now known as the "high-velocity method of washing" rapid sand filters. The difficulties to be overcome in the new method related to determining the velocity at which wash water could be forced up through the sand bed without disturbing the gravel layer, and yet produce a proper agitation of the sand; to the distance to place the waste troughs above the normal sand level in order to prevent loss of sand while washing; to the way in which the wash water should be first applied to prevent a mixing of the dirty top layers with the cleaner sand below by local eruptive discharges of the wash water at points of less resistance; to the proper length of the washing period, and consequently to the volume of wash water which would be required; and lastly to the effectiveness of the whole process as shown by the absence of any evidence of the accumulation of sediment within the filter bed.

The following quotation from the published record of this experimental work will serve to show the results of these studies of the author:

"The following conclusions appear to be justified by the result of this series of experiments:

"1. That the application of wash water to a filtering bed composed of Ohio River sand, having an effective size of 0.33 mm., at the rate of 2.0 cu. ft. per square foot per minute, produces an effective and satisfactory agitation of the sand.

"2. That the application of wash water at this rate thoroughly cleans the sand bed of any clay that may have been carried down into it.

"3. That sand is carried over the edge of a gutter 31 in. above the sand line in too large quantities when the wash water is applied at a rate of over 2.5 cu. ft. per square foot per minute.

"4. That at a rate of 2.5 cu. ft. per square foot per minute a small amount of sand is carried over a gutter 31 in. above the sand line, but the amount is probably negligible.

"5. That over 96 per cent. of the sand carried over a gutter 31 in. above the sand line at a rate of 2.0 cu. ft. per square foot per minute is less than 0.45 mm. in diameter.

"6. That a distance of 30 in. between the sand line and the edge of the gutter will probably prevent the loss of appreciable amounts of sand, when the wash water is applied at rates of less than 2.5 cu. ft. per square foot per minute." (Report of the Chief Engineer, Commissioners of Water-works of Cincinnati, Ohio.)

The successful use of this method of washing filters for over 8 years at the Cincinnati filtration plant, and its gradual adoption by other plants establishes its reliability and usefulness. Its advantages lay in the absence of any need for the entire apparatus required where compressed air is used, and which necessarily consists of air compressors, storage tanks, delivery piping, and sometimes a special distribution system of piping; in the greater simplicity of a single operation in place of two; in the fact that the agitating and washing are taking place at the same time and hence are more effective; and in the shorter period required to wash the filter, thereby lengthening the time a filter may be kept in effective service. The disadvantage of the method is chiefly in the somewhat larger sizes of pipes required in the wash-water distribution system, and which are necessary in order to convey the relatively large volume of wash water used in a period of usually less than 5 min. The method requires no greater percentage of wash water than does the method using compressed air and a lower rate of application of the wash water.

Mixing of the Sand and Gravel.—In the early experiments carried on by the author, the fear of mingling the sand and gravel by the application of the wash water at a high velocity, and the fact that such so-called "inversions" of filter beds were well known to have taken place in filters where low-velocity methods of washing were used, led to the use of a brass wire screen between the gravel and sand layers. This screen was fastened in place, rigidly holding down the gravel and preventing its upward movement, while allowing free motion to the sand. The screen had openings of such size (100 meshes per square inch) that the gravel could not pass through them, but permitted the sand to do so.

The screen fully demonstrated its effectiveness in the experimental work, and was consequently adopted as a part of the design of the large filter tanks of the Cincinnati filtration plant. This screen was made of brass and contained over 70 per cent. of copper and about 30 per cent. of zinc. Within 2 or 3 years after the filters were put into service breaks in the screen became more or less frequent. These were due in part to imperfect methods of fastening, but principally to actual corrosion of the brass. Continued corrosion of the brass further weakened the cloth so that breaks became quite frequent.

The remedy sought, and confirmed by experiment, consisted in removing the screen and substituting a heavier and deeper layer of gravel. The original gravel layer of 7.5 to 8 in. was deepened to 14 in., the lower layers or stones being composed of washed Ohio River gravel, the sizes of the stones of which varied from 1 to 2 in. in diameter. The depths of the several layers were made as follows in the reconstruction of the filter beds:

Reconstructed Filter Beds of Cincinnati Filter Plant[1]

	Depth of layer, inches	Gravel passing screen with mesh of, inches	Gravel retained on screen with mesh of, inches
1st layer.....	2		1
2d layer......	2	1	¾
3d layer.......	3	¾	½
4th layer....	4	½	¼
5th layer....	3	¼	...

The depth of the sand layer is from 28 to 30 in. The sand originally had an effective size of 0.34 mm., and a uniformity coefficient of 1.5 to 1.6. The sand has become somewhat coarser by washing and now has an effective size of 0.38 mm., and a uniformity coefficient of 1.35.

Loss of Head in Washing and Flotation of Sand.—From a series of experiments with an experimental filter, the losses of head obtained in washing filter beds of various depths, and the height of flotation of the sand, were observed, and are tabulated in the accompanying table. The gravel used was washed Ohio River gravel, consisting of rounded pebbles of varying sizes and placed in graded layers. The sand was also from the Ohio River and

[1] *Eng. Record*, vol. 71, p. 581.

had effective sizes of 0.31 and 0.41 mm., and uniformity coefficients of 1.45 and 1.41 respectively.

Experimental Data on Loss of Head in Washing Filters by the High-velocity Method[1]

Test number	Depth of gravel, inches	Depth of sand, inches	Upward velocity of wash water, feet per minute	Total head applied under filter, feet	Static head, feet	Loss of head in feet of water				Rise of sand, inches
						Through filter	Through strainer plate	Through gravel	Through sand	
4†	7½	20	1.0	8.33	6.40	1.93	0.58	0.06	1.29	2.5
4†	7½	20	1.5	9.00	6.42	2.58	1.20	0.09	1.29	4.9
4†	7½	20	2.0	9.92	6.43	3.49	2.05	0.14	1.30	7.3
5†	7½	25	1.0	8.70	6.40	2.30	0.60	0.06	1.64	2.8
5†	7½	25	1.5	9.42	6.42	3.00	1.22	0.09	1.69	6.3
5†	7½	25	2.0	10.29	6.43	3.86	2.03	0.14	1.69	9.8
6†	7½	30	1.0	9.00	6.40	2.60	0.60	0.06	1.94	2.6
6†	7½	30	1.5	9.81	6.42	3.39	1.30	0.09	2.00	6.6
6†	7½	30	2.0	10.63	6.43	4.20	1.99	0.14	2.07	10.7
7	7½	20	1.0	8.40	6.40	2.00	0.50	0.06	1.44	2.7
7	7½	20	1.5	9.07	6.42	2.65	1.12	0.09	1.44	5.3
7	7½	20	2.0	9.93	6.43	3.50	1.90	0.14	1.46	7.9
8	7½	25	1.0	8.75	6.40	2.35	0.50	0.06	1.79	3.7
8	7½	25	1.5	9.42	6.42	3.00	1.10	0.09	1.81	6.8
8	7½	25	2.0	10.27	6.43	3.84	1.90	0.14	1.80	9.8
9	7½	30	1.0	9.10	6.40	2.70	0.50	0.06	2.14	4.2
9	7½	30	1.5	9.76	6.42	3.34	1.07	0.09	2.18	7.8
9	7½	30	2.0	10.57	6.43	4.14	1.80	0.14	2.20	11.8
11	14	20	1.0	8.54	6.40	2.14	0.60	0.13	1.41	2.9
11	14	20	1.5	9.38	6.42	2.96	1.32	0.20	1.44	5.7
11	14	20	2.0	10.41	6.43	3.98	2.22	0.28	1.48	8.5
12	14	25	1.0	8.80	6.40	2.40	0.60	0.13	1.67	3.7
12	14	25	1.5	9.70	6.42	3.28	1.27	0.20	1.81	7.5
12	14	25	2.0	10.75	6.43	4.32	2.20	0.28	1.84	11.4
13	14	30	1.0	9.22	6.40	2.82	0.57	0.13	2.12	4.7
13	14	30	1.5	10.03	6.42	3.61	1.22	0.20	2.19	9.2
13	14	30	2.0	11.10	6.43	4.67	2.20	0.28	2.19	13.7
15	18	20	1.0	8.70	6.40	2.30	0.62	0.15	1.53	3.0
15	18	20	1.5	9.53	6.42	3.11	1.32	0.22	1.57	6.2
15	18	20	2.0	10.58	6.43	4.15	2.23	0.32	1.60	9.4
16	18	25	1.0	9.02	6.40	2.62	0.64	0.15	1.83	4.1
16	18	25	1.5	9.87	6.42	3.45	1.33	0.22	1.90	7.7
16	18	25	2.0	10.92	6.43	4.49	2.20	0.32	1.97	11.3
17	18	30	1.0	9.40	6.40	3.00	0.62	0.15	2.23	4.9
17	18	30	1.5	10.25	6.42	3.83	1.30	0.22	2.31	10.2
17	18	30	2.0	11.22	6.43	4.79	2.10	0.32	2.37	15.6*

† Brass wire screen in place. * Fine sand passed over wash-water troughs.

The relatively low losses of head through the gravel layers and the comparatively uniform losses of head through the sand for any

[1] J. W Ellms and J. S. Gettrust: "A Study of the Behavior of Rapid Sand Filters Subjected to the High-velocity Method of Washing." *Proc.* Am. Soc. C. E., January, 1916.

given depth of the latter, even for variations in the velocity of the wash water from 1 to 2 ft. per minute, are of interest in this table, as well as the greater relative increase in the height to which the sand floated as the velocity of the wash water was increased.

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CHAPTER XXVI

THE PHYSICAL AND CHEMICAL CHANGES PRODUCED BY THE APPLICATION OF CHEMICAL COAGULANTS AND BY THE SUBSEQUENT FILTRATION OF THE TREATED WATER

No intelligent operation of rapid sand filters is possible without a thorough knowledge of the chemistry of coagulation. The physical as well as the chemical characteristics of the water being treated must also be known, if the best purification results are to be obtained. Certain of the physical features of the coagulation of suspended impurities in water have been discussed in a previous chapter. The present chapter will deal more fully with some phases of the subject relating to filtration, and to the chemical reactions involved in the application of coagulants.

Coagulation.—The coming together in small flocculent masses of the finely divided suspended particles in a turbid water is induced by the addition of certain chemical salts, which react either with the salts naturally dissolved in the water or with alkaline bases that may be added with the coagulating chemical. The two coagulating chemical salts commonly used in treating waters are aluminum sulphate and ferrous sulphate. From the former, aluminum hydroxide is produced, and from the latter, ferric hydroxide. The gelatinous properties of these two hydroxides, and the ease with which they may be formed in the water, give to them their value for water-purification purposes.

A portion of the coagulated impurities are precipitated in preliminary settling basins. The balance must be removed by the filters. It is evident that once the flocculent masses have been formed, they should not be broken up by agitation. In consequence, after a thorough mixing of the applied chemicals with the water has been effected, the subsequent flow of the water should be at a low velocity, and as free as possible from any form of agitation. Spilling a coagulated water into a filter tank is not, therefore, good practice, since it breaks up the flocculent

matter, and also tends to keep the surface of the sand bed in the immediate vicinity of the inlet washed free of its colloidal coating.

Amount of Coagulated Sediment in Water Applied to Filters.—In the chapter on the efficiencies of settling and coagulating basins, a number of tables were given showing the residual turbidity of water passing from these basins. The range in the turbidity of the coagulated water, which was to be applied to rapid sand filters, varied from practically zero to 50 parts per million. The quantity of sediment which a filter will successfully handle obviously depends upon several factors; namely, the quantity and character of the colloids remaining in the water, the amount and kind of colloidal coating on the sand grains of the filter bed, the fineness and depth of the sand bed, and the rate of filtration. The larger the amount of coagulant applied, the thicker and more resistant will be the gelatinous coating on the sand surface. The lower the temperature which a water reaches, the less the retentive capacity of the colloidal hydroxide for sediment appears to be. The greater the rate of filtration, the farther into the sand bed will the minute particles be carried, and the sooner the filter bed will become overcharged. Finally, the character of the sediment itself plays an important part, since the finer the colloidal particles are, the more difficult it becomes to restrain them from passing through the filter bed.

Certain kinds of organic matter in the water materially assist the applied coagulant in preventing the passage of suspended particles. To illustrate, the author has found that with rates of filtration of 125,000,000 gal. per acre daily, usually not more than 15 to 20 parts per million of suspended matter produced by the clay particles found in the Ohio River water can be successfully filtered through a sand bed 30 in. deep, the sand of which has an average effective size of 0.38 mm. On the other hand, it has been observed with these same filters, that a turbidity of 100 parts per million in the applied water, resulting from a rise in the Ohio River during mid-summer, was filtered without any difficulty whatsoever.

The greater retentive capacity of the sand bed during the summer months is probably due to the organic colloids produced by the growth and death of microscopic plants which flourish in the river water during the warmer months of the year and which materially assist the chemically produced colloids in the filtra-

tion process. During periods of muddy water, this is obviously an advantage as it materially reduces the amount of chemical coagulant which would otherwise be required. On the other hand, the presence of these natural organic colloids, when the water is quite free from suspended matter, is detrimental to the filtration process, since their gelatinous character unmodified by clay particles, causes them to clog the bed too quickly, and increases the amount of wash water required to clean the bed.

Effect of Temperature on Coagulation.—In those climates where the temperature of the treated water during the cold season of the year reaches nearly to the freezing point, the coagulation of suspended matter during this period is imperfectly accomplished. The diminished efficiency appears to arise from some modification of the gelatinous properties of the chemical colloids produced, and which may be the result of a retardation of the chemical reactions involved, and the formation of smaller flocs having a diminished adsorptive power.

The practical effect of the low temperatures of the water upon the amounts of coagulant required is well illustrated by the curves in the accompanying cuts. These curves were prepared by Mr. H. W. Streeter, from observations made while operating a rapid sand filter plant, and are here published with his permission for the first time. Mr. Streeter's comments on these diagrams (Fig. 115) are as follows:

"These curves are based upon a series of diagrams prepared from data taken from daily records, and from many observations secured during about a year's operation of the plant. The diagrams shown were drawn for typical winter, spring and autumn, and summer conditions. Over each period a record of the temperature of the water was kept, and the mean was computed for each period. The variation in water temperatures for each period was not more than 5° from the mean.

The following notation was employed in plotting:

T_r = turbidity of raw water in parts per million.
T_c = turbidity of coagulated water in parts per million.

"Each line represents a certain value of T_r, and the amounts of aluminum sulphate necessary to give a coagulated water of a given turbidity (indicated by the value of T_c for each chart) are indicated for different temperatures of the water. The time of sedimentation, following the application of the coagulant, was about 5 hr.

"The curves are most poorly defined for extremely low and high raw

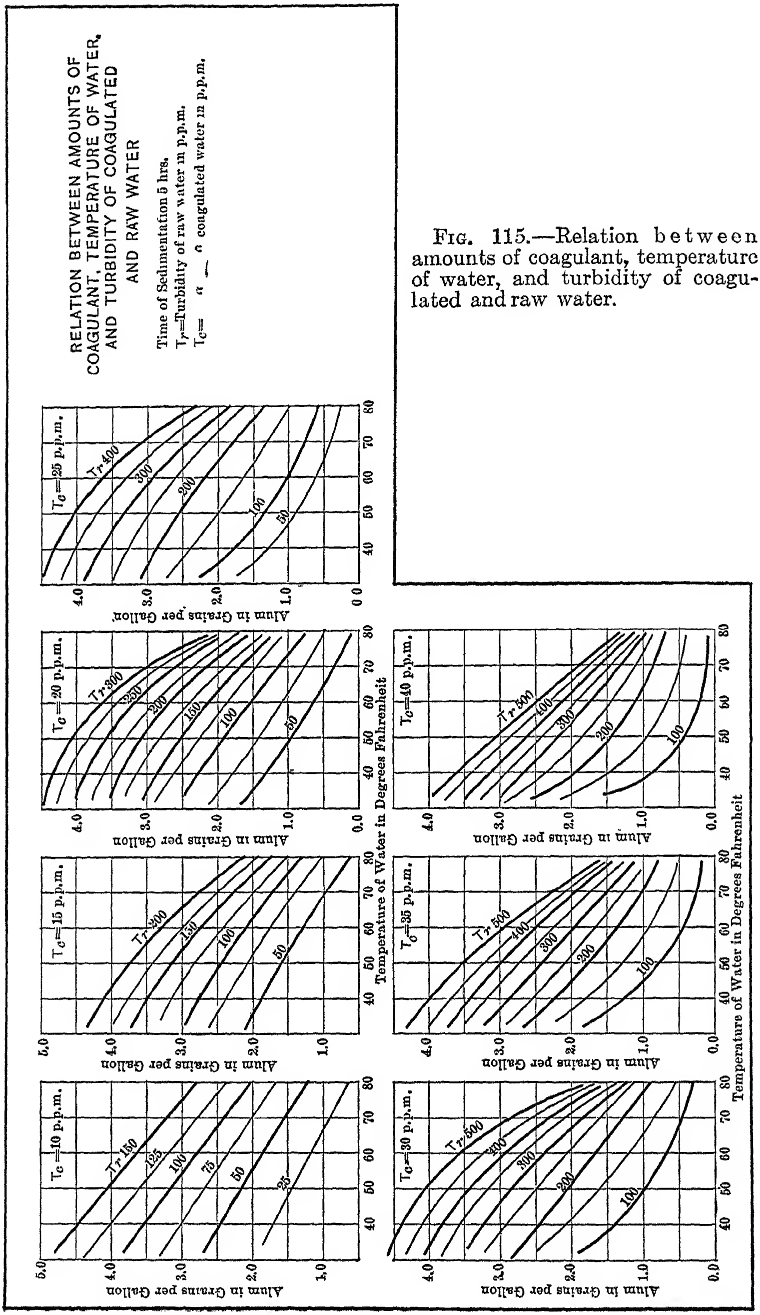

FIG. 115.—Relation between amounts of coagulant, temperature of water, and turbidity of coagulated and raw water.

water turbidities, and for very high applied water turbidities, since these were the conditions least frequently encountered.

"It should also be stated that the particular values obtained were based upon local conditions from which there might be considerable variation in other plants, particularly as regards the size and arrangement of the basins, their depth, their method of operation, and the method of applying the coagulant. For circumstances differing from those under which the values were obtained, results might not agree closely. The curves bring out, however, some very interesting relations between the amounts of coagulant required to produce a given degree of clarification at different water temperatures.

"The curves were found to be of great practical service in 'setting coagulant feeds' to meet sudden changes in the condition of the raw water.

"The influence of the variations in the character of the suspended matter in cold and in warm seasons, particularly when the raw water turbidity was high, may be a factor affecting the general trend of the curves, aside from the effect of the temperature of the water."

Chemical Reactions Involved in Coagulation.—When a water which is naturally alkaline from the presence in solution of the bicarbonates of calcium and magnesium, is treated with a solution of aluminum sulphate, a reaction occurs which may be illustrated as follows:

1. $Al_2(SO_4)_3$	+	$3H_2Ca(CO_3)_2$	$\leftrightarrows$	$3CaSO_4$	+	$Al_2(OH)_6$	+	$6CO_2$
Aluminum Sulphate		Bicarbonate of Calcium		Calcium Sulphate		Aluminum Hydroxide		Carbon Dioxide
Soluble		Soluble		Soluble		Insoluble in water, coagulum desired		Soluble

The reaction is precisely analogous if magnesium bicarbonate reacts with the aluminum sulphate instead of the calcium bicarbonate. In natural waters both of the carbonates of calcium and magnesium are held in solution principally by an excess of dissolved carbon dioxide. If insufficient amounts of naturally alkaline carbonates are present, then alkali must be added. This is usually provided by introducing a solution of sodium carbonate (soda ash) with the coagulating chemical. Caustic lime is sometimes employed with aluminum sulphate, but has the disadvantage of making the water harder by 7.7 parts per million for each grain of aluminum sulphate neutralized. The reaction with sodium carbonate is as follows:

$$2.\ Al_2(SO_4)_3 + 3Na_2CO_3 + 3H_2O \rightleftarrows 3Na_2SO_4 + Al_2(OH)_6 + 3CO_2$$

Aluminum Sulphate	Sodium Carbonate	Water	Sodium Sulphate	Aluminum Hydroxide	Carbon Dioxide
Soluble	Soluble	Molecules reacting	Soluble	Insoluble in water	Soluble

When ferrous sulphate or copperas is employed as the chemical coagulant, the reactions are somewhat different from those with aluminum sulphate. The iron being in the ferrous state, that is, in a condition in which it is capable of taking on more oxygen, abstracts the latter from the air dissolved in the water. Since it is ferric hydroxide, or the more highly oxidized iron compound that is the real coagulant, this oxidizing action must be favored and assisted. The best and cheapest method thus far found to assist this oxidation is by the addition of caustic lime. The reason for using this caustic lime may be better understood if the reactions involved are first given. Assuming that the sulphate of iron is added to the water before the caustic lime, then the following reaction occurs:

$$3.\ FeSO_4 + H_2Ca(CO_3)_2 \rightleftarrows CaSO_4 + H_2Fe(CO_3)_2$$

Sulphate of Iron	Bicarbonate of Calcium	Calcium Sulphate	Bicarbonate of Iron
Soluble	Soluble	Soluble	Slightly soluble

If caustic lime is now added to the water the following reaction takes place:

$$4.\ H_2Fe(CO_3)_2 + 2Ca(OH)_2 \rightleftarrows Fe(OH)_2 + 2CaCO_3 + 2H_2O$$

Bicarbonate of Iron	Calcium Hydroxide	Ferrous Hydroxide	Calcium Carbonate	Water reacting
Slightly soluble	Slightly soluble	Very slightly soluble	Very slightly soluble	

$$5.\ 4Fe(OH)_2 + O_2 + 2H_2O \rightleftarrows 2Fe_2(OH)_6$$

Ferrous Hydroxide	Oxygen	Water	Ferric Hydroxide
Very slightly soluble	From air dissolved in water	Reacting molecules	Insoluble in water; coagulum desired

By reference to reaction No. 3 it will be noted that the iron and calcium have exchanged places, and that a bicarbonate of iron is formed. This latter compound like the bicarbonate of

calcium is slightly soluble in water, principally because of the carbonic acid to which it is somewhat loosely bound. Its tendency to hold the iron in solution seems to produce a retarding effect on the oxidation of the iron to the ferric condition. In order, therefore, that the iron may become oxidized more quickly, the caustic lime is introduced to remove this loosely bound carbonic acid, as shown by reaction No. 4. The ferrous hydroxide is easily oxidized to ferric hydroxide by the oxygen dissolved in the water. Reaction No. 5 illustrates this change.

Even if no alkali is introduced, the instability of the bicarbonate of iron is shown by the partial hydrolysis of the compound, with the resultant formation of ferrous hydroxide. The reaction is markedly reversible.

6. $H_2Fe(CO_3)_2$	+	$2H_2O$	$\rightleftarrows$	$Fe(OH)_2$	+	$2H_2CO_3$
Bicarbonate of Iron		Reacting molecules of water		Ferrous Hydroxide		Carbonic Acid, or CO_2 plus one molecule of H_2O
Slightly soluble				Very slightly soluble		Soluble

The free carbon dioxide (CO_2) is liberated by violent agitation combined with aeration; and it is possible to hasten the above reactions (Nos. 6 and 5) and thereby avoid the use of an alkali. This method has been proposed, but the difficulties of applying the principles in practice on a large scale have not as yet been overcome.

Assuming that the lime is added before the ferrous sulphate, as is sometimes the practice, the free carbonic acid and the bicarbonates of calcium and magnesium react with the caustic lime as follows:

7. $H_2Ca(CO_3)_2$	+	H_2CO_3	+	$2Ca(OH)_2$	$\rightleftarrows$	$3CaCO_3$	+	$4H_2O$
Bicarbonate of Calcium		Free Carbonic Acid		Calcium Hydroxide		Calcium Carbonate		Water reacting
Slightly soluble		Soluble		Slightly soluble		Very slightly soluble		

The completeness of reaction No. 7 depends upon the amount of caustic lime added. In water-softening plants this reaction is usually carried far enough to remove all of the free and half-bound carbonic acid, and may and often is carried even farther, if it is desired to precipitate as hydroxide the magnesium of the

salts of this latter element. Under these conditions, there only remains to react with the ferrous sulphate that may be added, the small amount of calcium or magnesium carbonate which will remain in solution in a water even when carbonic acid is absent. If by chance an excess of caustic lime has been used, the reaction with the ferrous sulphate is, of course, direct.

It is evident, therefore, that there may be in solution to react with the ferrous sulphate applied, bicarbonates only, or bicarbonates and monocarbonates,[1] or monocarbonates and calcium hydroxide. Where there exist only monocarbonates, or monocarbonates and calcium hydroxide, the reactions may be expressed as follows:

$$8.\quad \underbrace{\underset{\text{of Iron}}{\underset{\text{Sulphate}}{FeSO_4}}}_{\text{Soluble}} + \underbrace{\underset{\text{Carbonate}}{\underset{\text{Calcium}}{CaCO_3}}}_{\text{Very slightly soluble}} \rightleftarrows \underbrace{\underset{\text{of Iron}}{\underset{\text{Carbonate}}{FeCO_3}}}_{\text{Very slightly soluble}} + \underbrace{\underset{\text{Sulphate}}{\underset{\text{Calcium}}{CaSO_4}}}_{\text{Soluble}}$$

$$9.\quad \underbrace{\underset{\text{of Iron}}{\underset{\text{Sulphate}}{FeSO_4}}}_{\text{Soluble}} + \underbrace{\underset{\text{Hydroxide}}{\underset{\text{Calcium}}{Ca(OH)_2}}}_{\text{Slightly soluble}} \rightleftarrows \underbrace{\underset{\text{Hydroxide}}{\underset{\text{Ferrous}}{Fe(OH)_2}}}_{\text{Very slightly soluble}} + \underbrace{\underset{\text{Sulphate}}{\underset{\text{Calcium}}{CaSO_4}}}_{\text{Soluble}}$$

$$10.\quad \underbrace{\underset{\text{of Iron}}{\underset{\text{Carbonate}}{FeCO_3}}}_{\text{Very slightly soluble}} + \underbrace{\underset{\text{of Water}}{\underset{\text{Reacting molecules}}{2H_2O}}} \rightleftarrows \underbrace{\underset{\text{Hydroxide}}{\underset{\text{Ferrous}}{Fe(OH)_2}}}_{\text{Very slightly soluble}} + \underbrace{\underset{\text{Acid}}{\underset{\text{Carbonic}}{H_2CO_3}}}_{\text{Soluble}}$$

The oxidation of the ferrous hydroxide proceeds as shown by reaction No. 5. Theoretically each grain of ferrous sulphate requires 0.5 part per million of dissolved oxygen to effect oxidation. This amount is very small, and no water utilized for a water supply is likely to have less than 10 to 15 times this amount as a rule.

Effects of Chemical Coagulants on Dissolved Constituents of Water.—From the foregoing reactions it is apparent that certain changes are produced in the dissolved constituents of a

[1] Monocarbonates may be defined as those compounds which contain no excess of carbonic acid over and above that directly combined with the base to form the normal salt.

water by the addition of the coagulating chemicals. The aluminum, *i.e.*, the metal is precipitated as the hydroxide, but the sulphuric acid with which it was combined remains dissolved in the water combined with the alkaline earth base. Analogous results are obtained with the ferrous sulphate. The iron is precipitated as the hydroxide, but the sulphuric acid remains dissolved. In both cases the acid unites with the calcium and magnesium in the water to form a sulphate. The carbonic acid liberated either remains dissolved in the water, or is precipitated as calcium carbonate whenever caustic lime is provided to unite with it.

The practical effect upon the dissolved constituents of the water is, therefore, to replace carbonates of calcium and magnesium by sulphates of these elements, or in other words to convert the temporary hardness or alkalinity of the water into permanent hardness. The total hardness of the water is not modified, but the permanent hardness is increased in proportion as the temporary hardness or alkalinity is decreased. These general statements are strictly true only when the natural alkalinity of the water reacts with the applied chemical. If soda ash is used to assist the natural alkalinity of the water to decompose the applied sulphate of aluminum or sulphate of iron, or is present naturally, the sulphate of sodium that may be formed does not add to the hardness of the water. On the other hand, if calcium hydroxide is employed, the permanent hardness is increased.

Where calcium hydroxide is employed merely to accelerate the decomposition of sulphate of iron, and is not needed to reinforce the alkalinity of the water, its effect upon the water depends upon the amount used and on the original quantities of the bicarbonates of calcium and magnesium which were present. If the original alkalinity of the water is below 50 parts per million (expressed in terms of calcium carbonate), and the amount of calcium hydroxide added is just sufficient to neutralize the sulphuric acid of the applied chemical coagulant, there will be no reduction in the alkalinity of the filtered water; if there is more caustic lime added than is needed by the chemical coagulant, but the amount is still only sufficient to bring the total alkalinity of the water up to about 50 parts per million, then the alkalinity of the filtered water will be increased. This means, of course, that the total hardness of the water will be increased to the extent that the alkalinity has been increased, since the additional per-

manent hardness is produced only by the destruction of an equivalent amount of alkalinity.

On the other hand, if the alkalinity of the water due to bicarbonates of calcium and magnesium exceeds about 50 parts per million and calcium hydroxide is used, the effect will be to reduce the total hardness of the water, since carbonates of calcium and magnesium in excess of about 50 parts per million will not remain dissolved when deprived of the carbonic acid which normally holds them in solution. Reference to reaction No. 7 will show a theoretically complete reaction between the bicarbonate of calcium and calcium hydroxide. It is obvious that where not enough caustic lime is used to effect a complete conversion of the bicarbonates of calcium and magnesium to the monocarbonates of these bases, a mixture of bicarbonates and monocarbonates will be present. This is actually the case where calcium hydroxide is used merely as an aid to the decomposition of the ferrous sulphate, and where the total alkalinity is below about 50 parts per million.

In the preceding paragraphs, 50 parts per million of residual alkalinity in a lime treated water is only to be regarded as approximate, since a number of factors, such as temperature of the water and time of reaction affect the final amount of the calcium and magnesium carbonates that will remain dissolved. In practice, however, a hard water softened by means of calcium hydroxide will show after 6 to 10 hr. settlement about 50 parts per million of alkalinity. This figure is naturally higher than the more exact laboratory results given in succeeding paragraphs, and which are not obtained in practice.

Theoretical Reduction of Alkalinity.—The application of 1 grain per gallon of aluminum sulphate ($Al_2(SO_4)_3 18H_2O$), to a water naturally alkaline with calcium and magnesium carbonates, will reduce the alkalinity, when expressed in terms of calcium carbonate, 7.7 parts per million. One grain of sulphate of iron ($FeSO_4 7H_2O$) per gallon will reduce the alkalinity 6.16 parts per million. These theoretical reductions are not always produced in practice; first, because commercially chemicals are not produced of precisely the exact composition given in the formulas, either purposely or because of the difficulties connected with their manufacture on a commercial scale; secondly, because of the adsorption of the undecomposed chemical on the surfaces of colloidal particles in suspension; and thirdly, because of some

direct combinations between the applied chemicals and the dissolved constituents of the water.

For 1 grain of sulphate of iron only 0.2 grain of calcium oxide is required to effect the decomposition of the former, but considering the impurities in the lime as usually obtained, and the losses in preparing solutions, it is usually necessary to use about twice as much (0.4) for each grain of the iron sulphate used.

The theoretical reduction in alkalinity produced by sodium carbonate for each grain of aluminum sulphate applied is 8.1 parts per million of Na_2CO_3, and the amount of carbonic acid (CO_2) liberated is 3.4 parts per million. It has been claimed that not all of the aluminum sulphate is changed to hydroxide (see reaction No. 2) by sodium carbonate, but that some basic sulphate of aluminum is produced, which is somewhat soluble and which might pass through a filter bed. Mr. George C. Whipple, in a paper[1] on "Hot-water Troubles," in which he discusses the effect of alum-treated waters on iron piping, makes the following statement:

"It is possible, also, that where the amount of alkalinity in the raw water is very low the alum reaction does not go to completion but a certain small amount of aluminum sulphate remains in the water. Conductivity experiments made by Meville C. Whipple have shown that whereas the conductivity of hard water is increased by a very small amount when alum is added, the conductivity of relatively soft water is considerably increased by the addition of the same amount of alum. Thus, in one series of experiments, in which 1 grain per gallon of alum was added to solutions of calcium carbonate of different strengths, it was found that the increase in conductivity in the solution containing 100 parts per million of calcium carbonate was only about 2 per cent., whereas in the solution containing 50 parts per million it was 15 per cent.; in the solution containing 25 parts per millien, 20 per cent.; and in the solution containing 10 parts per million, 35 per cent."

Solubilities of Calcium and Magnesium Carbonates and Magnesium Hydroxide.—Calcium carbonate and magnesium carbonate are only slightly soluble in pure water, but at ordinary temperatures and pressure will dissolve to the extent of about 1 gram per liter of the former, and 20 grams per liter of the latter, if the water contains sufficient carbon dioxide. In the absence of carbon dioxide calcium carbonate is only soluble to the extent of about 13 parts per million. Magnesium carbonate is much more

[1] *Proc.* Am. Water-works Assn., 1911.

soluble than calcium carbonate in the absence of carbon dioxide, but its exact solubility appears difficult to determine, since it appears impossible to prepare the pure carbonate of magnesia, it being invariably basic, that is, it contains both the hydroxide and carbonate of magnesium. The solubility of magnesium hydroxide (MgO_2H_2), however, has been determined with considerable accuracy, and appears to be about 10 parts per million.

It has been shown by Whipple and Mayer[1] that lime water reacts more rapidly with magnesium bicarbonate than with calcium bicarbonate, and that this is due to the fact that the precipitate is colloidal in character in the former case, and crystalline in the latter, although colloidal at first for a short time. It requires some time for calcium carbonate to be completely precipitated in the crystalline form. This fact is of practical importance as it explains the so-called "after-deposits" of carbonate of lime on filter sand, in softened-water reservoirs, and in distribution pipe lines. This phenomenon is of more consequence in water-softening plants than in rapid sand filter plants, although this effect may always be noted to a greater or less degree, wherever caustic lime is employed. Practically the greater part of a precipitate of calcium carbonate separates out within 6 hr. at ordinary temperatures, but the reaction appears not to be complete for a great many hours thereafter. Filtration of a water supersaturated with these compounds of lime and magnesia through sand reduces the quantity in solution, as shown by a reduction in the alkalinity of the filtered water.

Two experiments given by Whipple and Mayer, in the paper cited, are sufficient to show the relative solubilities and reaction periods for both calcium carbonate and magnesium hydroxide, when bicarbonates of these elements are treated with lime-water solutions.

In the first experiment just enough lime water was used to neutralize all the free CO_2 and precipitate all the calcium carbonate.

The results obtained in precipitating magnesium hydroxide with lime water are complicated by the solubility of the calcium carbonate produced by the precipitation of the calcium of the

[1] George C. Whipple and Andrew Mayer, Jr.: "The Solubility of Calcium Carbonate and Magnesium Hydroxide and the Precipitation of These Salts with Lime Water." *Jour. Infect. Diseases*, Supplement No. 2, February, 1906.

Experiment No.	Time of standing in hours	Temperature, degrees C	Parts per million of $CaCO_3$
I	1 0	22	92
II	3 5	22	35
III	6.0	22	26

lime water. The results obtained in two parallel experiments made at 35°C., and for the same periods of time by precipitating a solution of magnesium bicarbonate and a solution of calcium bicarbonate with lime water, and taking the difference between the observed alkalinities of the solutions as the amount of dissolved MgO_2H_2, are given in the following table:

Time in hours	Alkalinity in parts per million		
	Calcium carbonate	Magnesium hydroxide and calcium carbonate	Difference (MgO_2H_2)
0.25	55.0	85	30.0
0.50	40.0	66	26.0
1.00	33.0	60	27.0
2.00	29 0	55	26.0
3 00	28 0	53	25.0
5 00	25 0	50	25.0
10 00	22.0	48	26 0
24.00	18.5	39	20.5
48.00	16.0	36	20.0

Vegetable Coloring Matter.—The colloidal particles which produce color in a water are derived from the vegetable matter with which the water has come into contact. The coloring matter is composed of compounds of humic, gallic and tannic acids, derived from the decomposition of vegetable matter in swamps and lowlands partially covered with water.

The adsorption of this coloring matter by other colloids is the practical method of removing it from a water. For this purpose aluminum sulphate is used, since the aluminum hydroxide resulting from its decomposition produces a colloid with adsorption properties that seem able to withdraw quite a proportion of the coloring matter from the water. Usually not all of the color can be thus removed, unless excessive amounts of aluminum sulphate are used.

Iron salts from which ferric hydroxide would normally be precipitated are not adapted for removing color, since they unite with the organic acids and tend to produce colloidal iron compounds which deepen instead of diminish the color. A color of 20 parts per million or less in a water is not objected to by most people for drinking purposes. Efforts to reduce highly colored waters to less than a color of 10 parts per million are not generally advisable on the score of cost, since large amounts of coagulant are necessary to remove this residual coloring matter.

Presence of coloring matter appears to prevent in some measure the theoretical reduction in the alkalinity of water treated with aluminum sulphate, in the same manner as does the presence of large quantities of colloidal clay. Mr. George C. Whipple, in commenting on this fact in his paper on "Hot-water Troubles," says:

"It has been frequently noticed that the reduction in alkalinity is somewhat less than the theoretical. Fuller found this to be true with the turbid waters of the Ohio River, and explained it as being due to the absorption of a certain amount of the sulphate of alumina by the clay. The writer has found it to be the case with colored waters and especially with soft colored waters. It is possible that here also it may be explained by the absorption of the alum by the colloidal organic matter present in the colored water, but as it occurs also with waters that have been filtered and the suspended matter removed, it is perhaps more reasonable to believe that some alum unites chemically with the organic matter. The higher the color of the water, the larger the amount of aluminum sulphate that is combined in this way. It was found that when the amount of alum added was insufficient to

Effect of Coloring Matter in Water on Reduction of Alkalinity

Color	Original alkalinity	
	10 P.p m.	75 P.p.m.
	Reduction in alkalinity for each grain per gallon of aluminum sulphate used	
P.p m.	P.p.m.	P.p.m.
0	6.6	8.0
50	5.3	6.6

Note.—Experiments made by adding 0.75 grain per gallon of aluminum sulphate to distilled water to which calcium carbonate had been added to give the desired alkalinity, and extract of leaves to give the desired color.

coagulate and precipitate the coloring matter, the reduction in alkalinity was low.

"From the observations and experiments it seems reasonable to believe that with colored waters a double reaction takes place, and that a small part of the alum unites with the organic matter directly, while the larger part reacts with the calcium carbonate in the manner above mentioned. The nature of this aluminum organic compound is not known. Possibly it exists as a colloid."

The results of one of Mr. Whipple's experiments are shown in table on page 344.

Oxygen Dissolved in Water.—All natural surface waters contain more or less air dissolved in them. The quantity varies with both the temperature and pressure. At 32°F. and atmospheric pressure, 63.8 parts per million of air are dissolved in water. At the same temperature and pressure 14.7 parts per million of oxygen are dissolved. At 86°F., the amount of air dissolved falls to 33 parts per million, and the oxygen to 7.6 parts per million. Approximately 1 part per million of dissolved oxygen by weight in 1,000,000 parts of water corresponds with 0.7 c.c. of oxygen per liter of water.

The amount of oxygen required to saturate water at a pressure of one atmosphere and at various temperatures is given in the following table:[1]

Quantities of Dissolved Oxygen in Parts per Million by Weight in Water Saturated with Air at the Temperature Given

Temperature, degrees C.	Oxygen	Temperature, degrees C	Oxygen
0	14.70	16	9.94
1	14.28	17	9.75
2	13.88	18	9.56
3	13.50	19	9.37
4	13.14	20	9.19
5	12.80	21	9.01
6	12 47	22	8.84
7	12.16	23	8.67
8	11.86	24	8.51
9	11.58	25	8.35
10	11.31	26	8.19
11	11.05	27	8.03
12	10.80	28	7.88
13	10.57	29	7.74
14	10.35	30	7.60
15	10.14	...	

[1] "Standard Methods of Water Analysis." Report American Public Health Association.

A water may be supersaturated with oxygen at any given temperature, or it may contain less than the normal amount. Gill[1] has shown that a water may be heated 10° above its saturation point for 24 hr.—in some cases 48 hr.—without giving up entirely the excess of oxygen. Depletion of oxygen in a water is usually due to its removal by oxidizable substances in solution or in suspension. Supersaturation is not uncommon even in surface waters. Stored and impounded waters that are in contact with much organic matter, as for example, with the deposited sediment from muddy waters, or with the unstripped soil of reservoir bottoms, are quite commonly lower in dissolved oxygen than they are capable of holding on account of the using up of the oxygen by the decomposing matter undergoing slow oxidation.

The depletion of a water's oxygen on account of the use of ferrous sulphate as a coagulant is so slight (0.5 part per million for each grain of ferrous sulphate used per gallon) that it may be disregarded. Aeration to restore oxygen to filtered waters, which have been purified with the aid of this chemical is not, therefore, necessary and is not practised. Aeration of water to remove obnoxious gases or volatile matter producing offensive odors is frequently required in water-purification processes, and is usually undertaken after the water has passed through the filters.

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4. George C. Whipple: "Hot-water Troubles." *Proc.* Am. Water-works Assn.
5. George C. Whipple and Andrew Mayer, Jr.: "Solubility of Calcium Carbonate and Magnesium Hydroxide and the Precipitation of These Salts with Lime Water." *Jour. Infect. Diseases*, Supplement No. 2, February, 1906.
6. "Standard Methods of Water Analysis." Report Committee American Public Health Association.
7. Aug. H. Gill: "Upon the Difficulty with Which Water Parts with Its Dissolved Oxygen." *Technology Quarterly*, vol. 5, No. 3, October, 1892.
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9. George F. Catlett: "Colloidal Theories Applied to Colored Waters." *Eng. Record*, June 3, 1916.

[1] Aug. H. Gill: "Upon the Difficulty with Which Water Parts with Its Dissolved Oxygen." *Technology Quarterly*, vol. 5, No. 3, October, 1892

CHAPTER XXVII

EFFICIENCY AND COST OF OPERATION OF RAPID SAND FILTERS

The highest efficiency in the operation of rapid sand filters can be obtained only when the filters are of correct design, and when they are handled intelligently. The more complete the mechanical equipment of the filters, the easier become their operation. But no degree of perfection thus far attained in this direction can be regarded as a substitute for skillful manipulation The latter must be based upon a thorough knowledge of the processes employed, and an appreciation of the limitations of those processes and of the equipment being used. Excellent results maybe obtained with badly designed and poorly equipped plants by skilled operators; on the other hand, very low efficiencies may be obtained in well-designed and equipped plants by unintelligent and careless management.

The proper preparation of the water for filtration is of the utmost importance, and this fact can not be too strongly emphasized. Overloading of filters, by the application of water containing too large an amount of imperfectly coagulated sediment, is a frequent cause for the production of poorly purified effluents. The fact that the ability of filters to bear overloading may vary from time to time makes the most careful observation of operating conditions imperative at all times.

Period of Service of Rapid Sand Filters.—The length of time a rapid sand filter may be kept in service is obviously influenced by several factors. These are, as previously noted under "Loss of Head" in a preceding chapter, the rate of filtration, size of sand particles and depth of bed, character of applied sediment, entrainment of air within the bed, and the temperature of the water. In the gravity type of filter, where the total available head is usually in the neighborhood of 12 ft., it is apparent that the head may be either slowly or rapidly used up, depending upon the factor or combination of factors tending to clog the sand bed. Periods of service, therefore, actually vary from 1 hr. to over 500 hr., depending upon the particular factors involved.

If the rate of filtration is uniform, and is at or near the average for filters of this class, viz., 125,000,000 gal. per acre per day, the period of service will be much shorter, other conditions being equal, than if a uniformly low rate or a variable rate below the above-mentioned average rate is maintained. Beds of relatively fine sand (0.35 mm. effective size and under) will clog quicker than beds of coarse sand (0.35 to 0.60 mm. effective size). Deep beds or even quite shallow beds, which are washed by the "high-velocity method," whereby a preponderance of their finest particles are brought to or near the surface, will produce shorter periods of service, other things being equal, than shallower beds or those composed of coarser particles.

The chief factor influencing the length of the period of service is the character and amount of the suspended matter to be strained out of the applied water. As this material varies greatly both in quantity and kind in different localities, and even in the same locality at different seasons of the year, it causes the greatest variation in the service periods of the filters. For example, waters containing colloidal clay particles vary greatly at different seasons of the year in the amount of this kind of suspended matter. Under flood conditions the quantity in suspension is oftentimes very large, and sometimes the particles are very small and difficult to coagulate. Under such conditions the periods of service must necessarily be short if a clear effluent is to be maintained.

Variable Rates of Filtration.—The relation between a variable rate of filtration and the period of service is well illustrated by the following table taken from the Report of the Sewerage and Water Board of New Orleans for 1914 on the operation of their two filtration plants.

CARROLLTON FILTER PLANT

	Length of run in hours	Corresponding net rate of filtration, M. G per A. per 24 hr.	Corresponding quantity of water filtered, gallons	Corresponding per cent of wash water used	Corresponding length of run at rate of 125 M. G per A. per 24 hrs.
Maximum....	512.3	68.6	46,480,000	0.2	278.9
Minimum.....	22.7	69.3	2,125,000	1.9	12.8
Average. ...	129.6	82.1	14,170,000	0.5	85.0

ALGIERS FILTER PLANT

	Length of run in hours	Corresponding net rate of filtration, M G per A. per 24 hr.	Corresponding quantity of water filtered, gallons	Corresponding per cent. of wash water used	Corresponding length of run at rate of 125 M. G per A. per 24 hrs.
Maximum ..	353.8	83.4	5,057,000	0 2	236.0
Minimum..	27.3	58.5	280,000	0 6	13 5
Average.	188.2	88.9	2,870,000	0.4	134 0

The extremely long periods of service obtained in the operation of the filters of these two plants is unusual, and can be explained only by the character of the applied sediment, which is apparently much more easily strained out of the applied water than is generally the case.

The character of the water handled at the Carrollton plant, and the purification results effected, so far as the removal of turbidity is concerned, are shown in the following table:

TURBIDITIES

Year	Mississippi River water					Effluent grit reservoir				
	1910	1911	1912	1913	1914	1910	1911	1912	1913	1914
	Parts per million									
Maximum.....	1,700	1,400	2,200	1,900	2,400	1,450	1,250	1,950	1,800	2,000
Minimum......	55	150	190	120	60	55	130	160	130	50
Average...	550	500	750	675	600	450	425	775	600	575

Year	Effluent coagulation res.					Filters				
	1910	1911	1912	1913	1914	1910	1911	1912	1913	1914
Maximum.	525	280	1,700	650	425	1	0	5	5	3
Minimum	2	5	5	2	0	0	0	0	0	0
Average ...	32	32	40	28	32	0	0	0	0	0

The rapid sand filters of the Cincinnati filtration plant are operated at all times at a rate of 125,000,000 gal. per acre per day. They purify, like the New Orleans filters, a turbid water. Nevertheless the periods of service are much shorter on an average as may be seen from the following table:

PERIODS OF SERVICE AND PERCENTAGE OF WASH WATER OF CINCINNATI FILTERS

Year	Yearly average	
	Periods of service, hours	Percentage of wash water
1909	22.50	2.78
1910	15.56	3.50
1911	19.14	3.05
1912	20 32	2.51
1913	17 26	2.65
1914	19.50	2 24

The maximum and minimum monthly averages of the periods of service and of the wash water for the above years for the Cincinnati filters are as follows:

Year	Maximum monthly average			Minimum monthly average		
	Month occurred	Period service, hours	Per cent. wash water	Month occurred	Period service, hours	Per cent. wash water
1909	June	30.50	2.42	August	14.97	3.81
1910	June	24.19	2.49	August	7.13	6.42
1911	October	29.94	1.96	August	13.36	4.16
1912	August	32.38	1.34	March	10.44	3.80
1913	November	28.17	1.55	August	9.43	4.59
1914	September	35.39	1.26	June	10.12	3.36

Effect of Microscopic Plant and Animal Life on Periods of Service.—The presence in a water of microscopic plants and animals in any considerable amount, and especially at certain periods of their growth, have a very marked effect upon the periods of service of rapid sand filters. The comparatively short life cycle of these microörganisms, whereby they multiply with great rapidity, live for a few days or weeks and then disappear, is usually paralleled in the filter plant purifying the water by shortened periods of service of the filters. The clogging effect of these organisms is even greater than that of colloidal clay particles, although the former condition is less liable to be productive of poorly purified effluents than the latter.

In this connection it may be noted that the shortest periods of service of the Cincinnati filters occurred, as a rule, during

August. The minimum period of service in 1914 was in June. These low averages were caused by the clogging action of microscopic plants and of other colloidal organic matter. The minimum period of service in 1912, which occurred in March, was due to the high turbidity (360 parts per million) of the Ohio River water during that month. However, in August of this same year, the maximum period of service was reached, although the turbidity of the Ohio River water averaged 385 parts per million, or 25 parts per million higher than the average for March.

The difference in the efficiency of the filters during these 2 months can only be explained by the difference in the character of the applied sediment, and by the condition of the sand of the filters. The sediment carried by the flood waters during the first 3 or 4 months of the year contains little colloidal organic matter; the very fine clay particles are not readily coagulated, and have a tendency to pass through the filter bed and produce a turbid effluent. On the other hand, the turbid water passing down the stream in the summer months carries considerable organic matter, which materially assists the sand bed in straining out the suspended particles. Moreover, the sand grains acquire during the warmer months of the year a heavy gelatinous coating which is particularly effective in restraining the passage of finely divided clay particles. These differences in the character of the sediment and in the condition of the sand bed seem to offer a reasonable explanation for the varying efficiencies of the filters.

The effect of varying amounts of turbidity on the yield of filtered water for each foot loss of head is well illustrated in the accompanying curve (Fig. 116) kindly furnished the author by Mr. H. W. Streeter, which he plotted from data obtained by him in the operation of a rapid sand filter plant. The rapid increase in the yield after the turbidity of the applied water falls below 40 parts per million is quite marked.

Volume of Wash Water Required.—The proportion of the total volume of water filtered by a rapid sand filter plant, which is used for cleaning the filter beds, varies considerably. The longer the periods of service, the less proportionately will be the volume of wash water required. At the Carrollton plant in New Orleans, the percentage wash water required after one of the filter runs was but 0.2 per cent. of the total yield of filtered water for the entire period of service. The period of service of this filter was 512.3 hr.

The lowest monthly average percentage of wash water used by the entire Cincinnati filter plant was 1.26 in November, 1914; and the average for the 6 years recorded in the accompanying table is approximately 2.8 per cent. (see page 350). The maximum percentage of wash water as shown by the monthly averages for the Cincinnati plant for the above period was 6.42 in August, 1910.

The average percentage of wash water used during 1914 at the

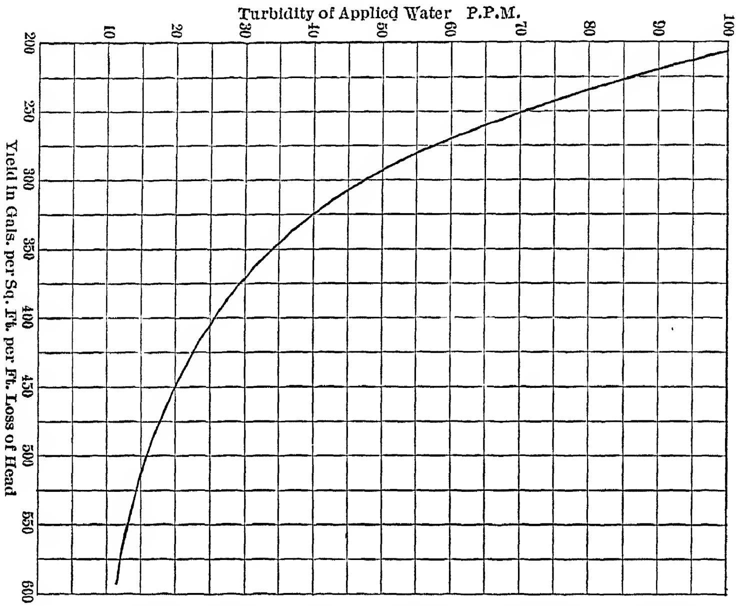


FIG. 116.—Average yield per foot loss of head for different turbidities of applied water.

Louisville, Ky., filter plant,[1] which like the Cincinnati plant purifies Ohio River water, was 3.16; the maximum monthly average was 5.48 per cent. (November), and the minimum 1.77 per cent. (April).

As an example of the percentage of wash water used in a rapid sand filter plant that purifies a water somewhat different in character from that of the Ohio River, the following table taken from the Report of the Board of Commissioners for the Water

[1] At the Louisville filter plant during 1914 a number of new filters were put into service which required considerably more wash water than did the old filters. The new filters required 4.55 per cent. of wash water and the old filters 2.97 per cent.; the average being 3.16 per cent.

and Lighting Department of Harrisburg, Pa., for 1912, is of interest.

Year	Turbidity, p p m.	Sulphate of aluminum, grains per gallon	Length of runs		Wash water, per cent.
			Hours	Minutes	
1906	110	0 95	18	20	2.0
1907	76	1 06	12	20	2 6
1908	52	1.09	13	29	2.6
1909	42	0 84	14	22	2.4
1910	19	0 61	18	40	2.2
1911	32	0 95	23	7	2.3
1912	59	0 77	21	54	2.3
Average.	54	0.895	20	20	2.34

At the purification works at Columbus, Ohio, the softening of the water is a prominent feature of the process. The following table compiled from the 1913 Report of the Division of Water of this city shows the relation between turbidity, color, amount of applied chemicals and percentage of wash water.

	Turbidity, p.p.m		Color, p p m		Chemicals applied, grains per gallon			Percentage wash water
1913	River water	Settled water	River water	Filtered water	Lime	Soda ash	Alum. sulph.	
Jan.....	198	16.0	34	7.0	6.3	2 9	2.9	2 7
Feb ..	32	3.0	24	3.0	7.7	6.0	1.8	2 1
Mar. ..	280	15 0	28	5.0	6.0	2 5	2.4	2.3
Apr....	178	12.0	38	6.0	6.6	2.6	2.0	2.4
May....	20	0.0	21	2.0	8.5	5 2	1.1	1.7
June....	21	0.0	28	5.0	8.9	5.9	0.9	1.2
July....	18	0.0	26	6.0	8.2	5.1	0.7	1.2
Aug ..	18	0.0	29	9.0	7.4	5.7	0.7	1.5
Sept....	16	2.0	19	8.0	7.4	7.5	0.8	1.6
Oct.....	14	11.0	16	5.0	8 3	8.6	1.7	1.5
Nov....	20	5.0	15	3.0	8.1	8.8	1.6	1.6
Dec....	11	7.0	21	6.0	8.2	11.7	1.7	2.1
	Average yearly results							
1909	86	...	42	8.7	7 9	3.8	1 76	4.1
1910	37	6.3	29	7.0	7.6	5.3	1.02	2.3
1911	68	3.2	28	7.0	7.5	4.3	1.57	1.3
1912	84	7.5	28	6.4	7.0	3 4	1.90	2.1
1913	69	6.0	25	5.0	7.6	6.0	1.50	1.8

Relation of Turbidity of Applied Water to Quantity of Wash Water.—The relation between the turbidity of the water applied to filters and the percentage of wash water required to cleanse them is well brought out by the accompanying curve prepared by Mr. H. W. Streeter from data secured during the operation of a rapid sand filter plant. His comments on the diagram (Fig. 117) are as follows:

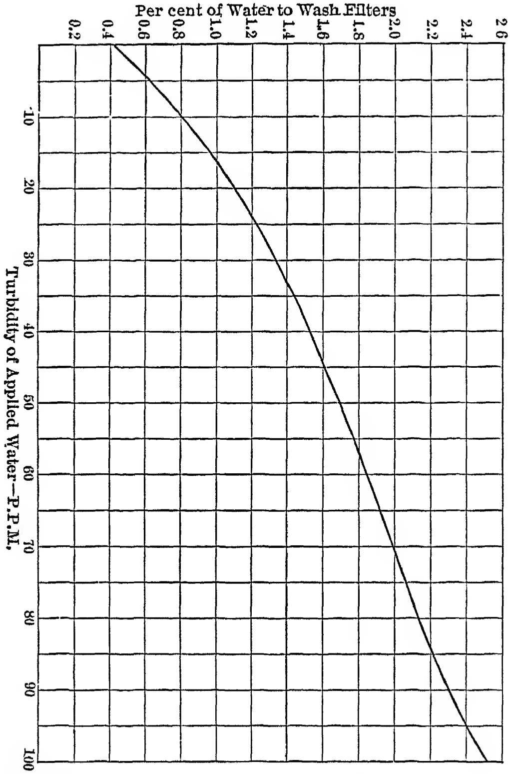


FIG. 117.—Relation between turbidity of applied water and amounts of water necessary to wash filter.

"The curve is based upon a year's daily operating records. The time required for the application of the wash water was very carefully recorded, and the rate of application was frequently checked by noting the rise of the wash water in the filter tank by means of a gage placed therein. The rate of application of the wash water was subject to very accurate control.

"The turbidity of the applied water was taken as the mean prevailing throughout the filter run which terminated with the washing being

considered. Usually the variation in the turbidity was slight for the comparatively short period involved.

"The curve is not considered as reliable for turbidities of applied water above 50 parts per million as for those below this turbidity. Below a turbidity of 50 parts per million there were more observations available and these lay along a well-defined line."

The following data obtained in the operation of the filtration plant at Little Falls, N. J., and tabulated by Mr. G. A. Johnson in his paper on "Present-day Water Filtration Practice"[1] is a good illustration of the efficiency of a rapid sand filter plant handling for the most part a highly colored water rather than one with much turbidity. The bacteria given in the original table are omitted.

1912	Sulphate aluminum, grains per gallon	Per cent. wash water	Turbidity				Parts per million, color			
			Average		Highest day		Average		Highest day	
			R	F	R	F	R	F	R	F
Jan....	0.74	1.7	7	0	9	0	32	5	38	8
Feb...	0.65	1.8	13	0	30	0	35	6	53	11
Mar...	0.98	2.7	26	0	118	0	52	6	92	15
Apr.....	1.07	2.8	8	0	13	0	41	6	46	9
May. .	1.46	3.3	8	0	13	0	55	7	66	11
June. .	1.60	3.3	10	0	12	0	56	8	65	11
July .	1.65	3.4	9	0	11	0	46	9	55	11
Aug. .	1.62	3.4	8	0	17	0	43	9	52	10
Sept....	1.47	3.1	7	0	10	0	41	8	64	10
Oct....	1.93	3.1	10	0	23	0	56	7	78	10
Nov...	3.16	3.0	10	0	22	0	61	14	71	22
Dec....	1.86	2.3	8	0	12	0	40	11	53	19

REMARKS: R = river water. F = filtered water.

Mr. Johnson comments as follows on this data:

"The records given in the last table show that while on one day in 1912 the turbidity of the water rose to 118 parts per million, the yearly average turbidity of the raw water was only 10 parts per million. The filtered water was free of turbidity at all times. The color of the unfiltered water reached a maximum of 92 parts per million in March, while the average for the year was 46 parts. The filtered water color averaged 8 parts for the entire year, and here it should be noted that at this plant the aim is to reduce the color so that the filtered water shall

[1] *Jour. Am. Water-works Assn.*, March, 1914.

not contain more than 10 or 15 parts. In November and December the color of the filtered water was slightly in excess of this upper limit for short periods."

The average amount of sulphate of aluminum used at the Little Falls plant during 1912 was 1.51 grains per gallon, or 216 lb. per million gallons of water treated. The average percentage of wash water was 2.8. The filters are washed with water applied at the rate of 8 gal. per square foot per minute, and compressed air is used for the agitation of the sand bed.

Hardness.—The effect of purification upon the dissolved salts causing hardness in a water is obviously dependent upon the kind and amount of coagulants and softening agents used. The addition of sulphate of iron or of aluminum to a water containing sufficient alkalinity to decompose them necessarily increases its content of sulphate of calcium and of magnesium, and hence its permanent hardness. If softening agents, such as lime and soda ash are applied to a water containing considerable quantities of salts of calcium and magnesium, a reduction in the dissolved hardening constituents may be effected. How much change in the hardness of the purified water is produced by a number of purification plants is shown by the following table given by Mr. George A. Johnson in the same paper quoted from above.[1]

HARDNESS OF RIVER WATERS AT VARIOUS PLACES BEFORE AND AFTER COAGULATION AND FILTRATION

City	Average for year	Kind of filter	Amount coag. used, grains per gallon	Parts per million				Increase of incrustants in filtered water due to the use of coagulants
				Total hardness		Incrustants		
				River	Filt.	River	Filt.	
Springfield, Mass...	1912	S	(*d*) 0.24	11	2	4	2	2
Little Falls, N. J...	1903	R	(*d*) 1.38	31	30	7	14	7
Louisville, Ky ...	1912	R	(*d*) 1.73	95	91	29	38	9
Cincinnati, Ohio..	1910	R	(*a*) 0.84					
			(*b*) 1.79	76	89	32	41	9
New Orleans, La..	1912	R	(*a*) 4.41	111	60	21	25	4
			(*b*) 0.33					
			(*a*) 7.50					
Columbus, Ohio....	1910	R	(*c*) 4 30	270	85	111	35	Softened
			(*d*) 1.57					

NOTE.—R designates rapid sand filters and S slow sand filters. (*a*) Lime; (*b*) Sulphate of iron; (*c*) Soda ash; (*d*) Sulphate of aluminum.

[1] G. A. JOHNSON: "Present-day Water Filtration Practice." *Jour. Am. Water-works Assn.*, March, 1914.

At the first four purification plants listed in this table, the use of chemicals is simply and only for the purpose of coagulation in order to prepare the water for filtration. At the last two plants, however, chemicals are used to effect a softening action in conjunction with those applied for coagulation purposes.

Bacterial Purification.—The efficiency of rapid sand filter plants in the removal of bacteria ranges from 90 per cent. to over 99 per cent. in well-operated plants. With few bacteria in the applied water, the percentage removal will not be as high as when the number is large. With large numbers of bacteria in the applied water, however, the number of bacteria actually passing through the filter bed may be and generally is much higher than where the number of bacteria in the applied water is low.

The following tables taken from the reports of the water department of Harrisburg, Pa., and Cincinnati, Ohio, indicate the bacterial content of the water at these two purification plants in various stages of the process, and the percentages removed.

HARRISBURG FILTRATION PLANT

Year	Bacteria per cubic centimeter			Percentage Removals by		
	River water	Settled water	Filtered water	Coagulation basin	Filters	Total
1906	12,372	3,228	94	73 91	97.06	99 24
1907	10,710	4,504	44	57.96	99.03	99.59
1908	4,949	1,662	19	66.43	98.86	99.62
1909	5,762	1,083	17	80.87	98.28	99.70
1910	7,843	62	5	99.02	91 94	99.94
1911	10,357	58	7	99.04	87.93	99.94
1912	5,115	35	2	99.32	94.29	99.97
Average	8,239	1,519	27	82.36	95.34	99.71

NOTE.—Calcium hypochlorite has been used since late in 1909, and is applied to the raw water as it enters the settling basin. This use of a disinfectant somewhat obscures the real efficiency of the plant so far as the effect of the coagulating chemicals and the filters are concerned; but results prior to 1909 are not thus affected.

CINCINNATI FILTER PLANT

	Average number of bacteria per cubic centimeter						
	1908	1909	1910	1911	1912	1913	1914
River water	7,000	9,300	8,900	13,790	11,130	16,100	16,500
Settled water	3,400	2,500	3,200	3,140	3,945	2,965	3,570
Applied water	700	475	750	776	898	465	670
Filtered water	100	75	75	39	26	56	36
Average percentage removal	98.5	99 2	99.2	99.7	99.8	99.7	99.8

NOTE.—Disinfectant used in filtered water during the first 3 to 6 months of each year beginning in 1911, but was not used prior to that year.

Some very complete data obtained in operating the New Orleans (Carrollton) filter plant are given below.

BACTERIA PER CUBIC CENTIMETER

	River					Effluent grit reservoirs				
	1910	1911	1912	1913	1914	1910	1911	1912	1913	1914
Maximum	41,000	8,500	32,000	20,000	9,000	50,000	8,500	19,000	30,000	8,500
Minimum	275	95	400	100	400	140	40	475	230	190
Average	4,200	2,400	3,300	1,900	1,800	3,500	2,000	3,400	1,700	1,700

	Effluent coag. reservoirs					Filters				
	1910	1911	1912	1913	1914	1910	1911	1912	1913	1914
Maximum	11,000	7,000	12,000	11,000	4,000	1,300	3,000	3,200	550	325
Minimum	12	5	23	10	13	1	2	2	1	1
Average	1,500	400	1,000	350	190	(a) 80 (b) 26	(a) 90 (c) 38	(a) 140 (c) 40	21	19

(a) For entire period.
(b) Excluding March and April.
(c) Excluding period of abnormal counts due to algæ growths, and use of hypochlorite of lime.

Efficiency in the Removal of the Bacillus Coli.—The identification of fecal bacteria in a water has become a part of the routine practice in the laboratories of water-purification plants; and the results obtained by tests of the raw and filtered waters are regarded as a measure of the efficiency of the purification process. In the absence of direct methods for enumerating organisms of this class, more or less indirect methods must be used. The

smaller the sample of water which gives a positive test indicating the presence of these organisms, the greater is its degree of pollution. On the other hand, the larger the sample in which positive tests are obtained, the less likelihood is there that it contains pathogenic organisms which may have been associated with the fecal bacteria. The tests for B. coli, therefore, may be used, if properly interpreted, to indicate the purity of the water tested, and hence may be made a measure of the efficiency of the purification processes.

Some data obtained in the operation of the filter plant at Louisville, Ky., during 1914, is of interest in this connection. Out of 2,190 samples of Ohio River water tested 1,762, or 80.5 per cent., gave positive evidence of the presence of the B. coli. Out of 2,195 samples of filtered water only 26, or 1.2 per cent., gave positive tests for this organism. These results were presumably obtained by testing 1-c.c. samples. Disinfectants were used 130.5 days during the year, and hence the figures given include the effect of both filtration and disinfection.

At the Columbus, Ohio, filtration and water-softening plant during the year 1913, 204 1-c.c. samples of the raw water gave a positive test for B. coli; and 2, 14 and 19 samples of the filtered water of 1-c.c., 10-c.c. and 50-c.c., respectively, also gave positive tests. The filtered-water results, the same as in the Louisville filter plant data, include the effect of the use of disinfectants.

The following table taken from the 1912 Report of the Board

TABLE SHOWING THE PERCENTAGE OF POSITIVE TESTS FOR B. COLI-COMMUNIS IN THE UNFILTERED, FILTERED AND TAP WATERS AT HARRISBURG, PA., FILTER, PLANT

Year	Unfiltered	Filtered	Tap
1906	71.87	2.73	4.95
1907	64.02	1.02	2.64
1908	65.72	1.07	1.62
1909	63.15	1.00	1.60
1910	55.44	0.17	0.82
1911	77.30	0.61	1.46
1912	46.90	0.81	2.81
Average for 7 years..........	63.49	1.06	2.27

NOTE.—These results are all on 1-c.c. samples. Disinfection of the coagulated water has been practised since late in 1909.

The following table gives the total number of tests, the number and percentage of positive tests on various quantities, and the B. coli index of the river and of the filtered water:

Month, 1914	River water											Filtered water							
	Bacillus coli										B coli index No per c c	Bacillus coli							B coli index No. per c c
	No. of test days	0.01-c c. tests			0.1-c c. tests			1.0-c c tests				No. of test days	1.0-c c tests			20.0-c c tests			
		Total No.	No. +	Per cent. +	Total No	No. +	Per cent +	Total No	No +	Per cent +			Total No.	No. +	Per cent. +	Total No.	No. +	Per cent +	
January	31	31	3	9 68	31	19	61 29	31	31	100.00	16 21	31	31	0	0 00	31	1	3 23	0.00281
February	28	28	3	10 71	28	15	53 57	28	28	100 00	15.46	28	28	0	0 00	28	1	3 57	0 00476
March	31	31	1	3 23	31	15	48.39	31	31	100 00	8 26	31	31	0	0 00	31	0	0 00	0 00032
April	30	30	3	10.00	30	16	53 33	30	30	100 00	14.80	30	30	0	0 00	30	0	0 00	0 00078
May	31	31	5	16.13	31	15	48 39	31	29	93 55	19 81	31	31	0	0 00	31	7	22 58	0 01516
June	30	30	1	3 33	30	4	13 33	30	24	80 00	5 00	30	30	0	0 00	30	1	3 33	0 00589
July	31	31	1	3 23	31	11	35 48	31	20	65.42	6.75	31	31	0	0.00	31	1	3.23	0 00484
August	31	31	9	29 03	31	16	51 61	31	29	93 55	31.71	31	31	1	3 23	31	6	19 35	0 94488
September	30	30	3	10 00	30	12	40 00	30	26	86.67	13 47	30	30	0	0 00	30	6	20 00	0 01567
October	31	31	6	19.35	31	18	58 06	31	29	93.55	23.58	31	31	2	6.45	31	16	51 61	0 08945
November	30	30	0	0.00	30	1	3 33	30	20	66 67	0 97	30	30	0	0.00	30	6	20 00	0 01611
December	31	31	4	12.90	31	28	90 32	31	31	100 00	20.74	31	31	1	3 23	31	26	87 10	0 07531
Total	365	365	39	...	365	170	...	365	328			365	365	4		365	71		
Average	...	...	...	10 63	...	...	46.43	.	.	89.88	14 73				1.08		. .	19.50	0 0230
Per cent time	100.00	100.00	10.69	...	100.00	46 58	. .	100 00	89 87	...	...	100 00	100 00	1 10		100 00	19 45	...	...

NOTE.—Positive presumptive tests of 1-c c samples were confirmed in the usual way all the results of other tests are presumptive, and were obtained with lactose bile

From Jan 1 to May 27, 438 samples of filtered water showed 92.69 per cent. of tests to be positive in 100 c.c before disinfection, and 23 97 per cent. positive in 100 c c. after disinfection.

of Commissioners of Harrisburg, Pa., shows the percentage of positive B. coli tests obtained in the raw and filtered waters and in the tap water for a period of 7 years.

The data obtained in the laboratories of the Cincinnati, Ohio, filtration plant as to the presence or absence of B. coli, are given in some detail in the Report of the Water Department of Cincinnati for the year 1914.

The following table shows the numbers of bacteria per cubic centimeter and the percentage of positive B. coli tests on various quantities of filtered water before and after the application of 0.1 part per million of chlorine:

Month, 1914	Bacteria per c.c.		Per cent. positive B. coli					
			1.0 c.c.		20 0 c.c.		100.0 c.c.	
	Before	After	Before	After	Before	After	Before	After
January. . .	300	26	3.23	0.00	58.06	3.23	94.62	16.13
February....	370	87	0.00	0.00	57.14	3.57	94.05	33 33
March......	305	39	0.00	0.00	12.90	0.00	77.53	3 23
April. . . .	80	13	16.67	0.00	83.33	0.00	97.78	7.78
* May.	75	22	14.81	0.00	70.37	18 52	100 00	55 55
Average. . .	226	37	6.94	0.00	56.36	5.06	92.80	23.20

* Chlorine applied from Jan. 1 to May 27, 1914.
Percentage removal of bacteria by disinfection, 83.63.

The general significance to be attached to these figures for B. coli, when used as a definite measure of purity of a water will be discussed in a succeeding chapter.

COST OF OPERATION OF RAPID SAND FILTER PLANTS

The cost of operating a rapid sand filter plant is obviously dependent upon the completeness of its equipment, upon the character of the water being treated, and on the skill with which the plant is handled. A poorly equipped plant will require more men to operate it than one well provided with labor-saving devices. A water which requires a large amount of chemicals to properly purify it is necessarily expensive to treat. Unintelligent management of both men and material is productive of high costs. The cost for labor and chemicals constitute the largest part of the total cost of operation and maintenance.

Mr. G. A. Johnson[1] has tabulated the operation and mainte-

[1] "Present-day Water Filtration Practice." *Jour. Am. Water-works Assn.*, March, 1914.

nance costs for a number of both slow sand and rapid sand filtration plants.

Year	City	Kind of filter	Average volume of water filtered daily, gallons	Cost of operation and maintenance per million gallons of water filtered
1911	Albany, N. Y.	Slow sand	20,000,000	$2.50
1912	Pittsburgh, Pa.	Slow sand	100,000,000	3 41
1911	Philadelphia, Pa.	Slow sand (*a*)	9,000,000	5.62
1911	Philadelphia, Pa.	Slow sand (*b*)	13,000,000	3.59
1911	Philadelphia, Pa.	Slow sand (*c*)	38,000,000	3 88
1911	Philadelphia, Pa.	Slow sand (*d*)	202,000,000	1.91
1912	Washington, D. C.	Slow sand	62,000,000	4.01
1912	Cincinnati, Ohio	Rapid sand	50,000,000	4.12
1911	Harrisburg, Pa.	Rapid sand	9,000,000	3.93
1912	Little Falls, N. J.	Rapid sand	30,000,000	3.20
1912	Louisville, Ky.	Rapid sand	25,000,000	3.48
1912	New Orleans, La.	Rapid sand	16,000,000	6.32
Weighted average,		Slow sand		$2.86
Weighted average,		Rapid sand		4.04

(*a*) Lower Roxborough; (*b*) Upper Roxborough; (*c*) Belmont; (*d*) Torresdale.

As an illustration of the effect upon costs of operation, where a difficult water is to be purified, the detailed figures given in the 1913 Report of the Division of Water of Columbus, Ohio, are of interest.

	Total cost	Cost per million gallons of water purified	Per cent. of total cost
Labor	$23,760.13	$3.78	23.5
Repairs to lands and buildings	260.07		
Repairs to machinery	32.00		
Repairs to other equipment	1,016.33		
Office and laboratory supplies	1,571.07		
Heavy chemicals	69,999.06	11.17	69.4
General supplies	2,707.74		
Oil, waste and machine supplies	169.71		
Miscellaneous	1,411.44		
Total	$100,927.55	$16.08	

NOTE.—Cost per million gallons based on 6,276,675,000 gal. of water delivered for consumption.

The cost of labor and chemicals constitute 92.9 per cent. of the total cost, leaving but 7.1 per cent. for all other purposes.

The Carrollton plant costs at the New Orleans filter plant for the year 1914 are also given in much detail, and are as follows:

CARROLLTON

OPERATION

	Total cost	Cost per million gallons
1. Labor, attendance and supervision.	$23,319 68	$2 86
2. Labor—unloading, crushing and storing chemicals...	1,510.25	0.19
3. Lime.....	14,745.93	1.81
4. Iron.......	2,532.96	0 31
5. Supplies, tools, petty cash, car fare, telephone, ice, stamps, bond premiums, etc................................	1,678.82	0.21
6. Machinists' labor furnished by pumping station..	114.95	0.01
7. Labor and material furnished by pumping station for power, heating and lighting....	1,300.00	0.16
Total cost of operation....	$45,202.59	$5.55
Credit for sale of filling and sand.. .	465.08	0.06
Net cost of operation..........	$44,737.51	$5.49

Betterments and Additions:

1. Labor (includes $582.97 machinists' labor)	$ 3,179.27
2. Material...	1,267.06
	$ 4,446.33

PARK AND GROUNDS

Care and Maintenance:

1. Labor—care of grounds, etc.	$ 3,879.90
2. Labor—watchman........	894.40
3. Labor and material furnished by pumping station for lighting........ .	550.00
4. Material and supplies.........	116.25
	$ 5,440.55

Betterments and Additions:

1. Labor.............................	$ 205.15
2. Material...........................	119.00
	$ 324.15

"All figures for cost are exclusive of interest and depreciation charges.

"All figures above for cost of purification per million gallons are based on actual quantity of water treated during year, as shown by corrected Venturi meter readings, viz., 8,147,000,000 gal., and are exclusive of all charges for high- and low-lift pumping.

"Cost of wash water, at $12.75 per million gallons used, based on amount of water pumped by high-lift pumps, including all pumping costs, was 7 cts. per million gallons of water filtered.

"Cost of all water for cleaning reservoirs was 3.6 cts. per million gallons of water filtered, excluding treated water wasted in draining reservoirs for cleaning.

"This makes the net cost of filtered water pumped $12.86 per million gallons.

"Total gross cost of delivering filtered water to the distribution system, at the plant, exclusive of interest and depreciation charges, based on quantity actually delivered, viz., 8,010,000,000 gal., was $13.06 per million gallons, to cover all costs of pumping and purification, or $14.33 per million gallons, if the betterments and care and maintenance of the park and grounds, etc., are included."

The purification plants at New Orleans and at Columbus, Ohio, not only clarify but soften the water. The cost of softening is high compared with the cost for clarification. The two plants on the Ohio River at Louisville, Ky., and Cincinnati, Ohio, are better examples of plants which clarify and filter the water without attempting to soften it.

COST OF FILTER OPERATION IN 1914 AT LOUISVILLE, KY.

	Total cost	Cost per million gallons
Superintendence and laboratory pay roll. .	$ 4,980.00	$0.50
Filter operators' pay roll..	8,250.00	0.83
Coagulant....	13,324.31	1.35
Wash water.	2,679.23	0.27
Heat, light and power.	2,095.96	0.21
Supplies...	426.77	0.04
Repairs...	353.17	0 04
Incidentals...	788.00	0.08
Total....	$32,897.44	$3.32

NOTE.—Volume of filtered water on which above calculations of cost per million gallons are based, was 9,892,887,622 gal.

A condensed table of the cost per million gallons for operating and maintaining the Cincinnati filtration plant for 7 years, based upon the volume of water delivered for consumption (not the total water filtered), is as follows:

Year	1908	1909	1910	1911	1912	1913	1914	Average for 7 years
Operation	$4 19	4 17	3.91	3.77	3.46	3.44	2.99	$3.70
Maintenance..	0.05	0.09	0.28	0.35	0.38	0.48	0.39	0.29
Total.......	$4.24	4 26	4 19	4 12	3.84	3.92	3 38	$3.99

Of these total costs per million gallons there were expended the following amounts for coagulating chemicals.

Year	1908	1909	1910	1911	1912	1913	1914	Average for 7 years
Cost per M. G.	$1.72	1.89	1.93	1.86	1.78	1.67	1.21	$1.72
Percentage of Total cost...	40.6	44.4	46.0	45.1	46.4	42.6	35.8	43.1

The cost of operating the Harrisburg, Pa., filter plant during the year 1912, as given in the yearly report of the Water Department, is quoted below.

"The operating expenses were $17,865.50, coagulant costing $4,390.95; coal, $1,171.09; supplies, $1,377.57; material and repairs, $1,687.10; oil and waste, $163.07; chemist and laboratory, $1,212.38; salaries, $7,854.59; and boiler repairs, $8.75. This makes the cost per million gallons $5.53, coagulant costing, $1.35; coal, 36 cts.; supplies, 42 cts.; repairs, 52 cts.; oil and waste, 5 cts.; chemist and laboratory, 37 cts.; and salaries, $2.46.

The fixed charges for interest, sinking fund and State tax on the filtration loan was $22,423.33, a cost per million gallons of $6.94.

Fixed Charges.—Beside the cost of operating and maintaining a filter plant, there should properly be charged against it the interest on the amount of capital invested and a reasonable charge for depreciation. The ordinary operation charges will range from $4 to $6 per million gallons, and all other charges, including interest, sinking fund and depreciation will add from 75 to 125 per cent. to the above costs. The whole cost of filtration will, therefore, probably range from $7 to $14 per million gallons.

References

1. Reports of the Sewerage and Water Board of New Orleans, La.
2. Reports of the Water Department of Cincinnati, Ohio.
3. Reports of the Division of Water, Columbus, Ohio.
4. Reports of the Board of Water and Light Commissioners, Harrisburg, Pa.
5. Reports of the Louisville Water Co.
6. G. A. Johnson: "Present-day Water Filtration Practice." *Jour. Am. Water-works Assn.*, March, 1914.
7. H. P. Letton: "Operation of Rapid Sand Filters." *Municipal Journal*, 1914.
8. George W. Fuller: "The Proper Operation of Filters." *Eng. Record*, vol. 58, p. 498.

CHAPTER XXVIII

DISINFECTION OF WATER SUPPLIES

The removal from a water of disease-producing organisms, as well as those that are harmless, by the methods described in the preceding chapters, is accomplished largely by settling and by mechanically straining or filtering them out of the water. The pathogenic organisms which by chance pass through a filter may or may not be injurious from a sanitary point of view, since their ability to infect depends in a great measure upon their number and virulency. Those water supplies which at present appear to be only slightly polluted, and which are still used unfiltered, may be at times dangerous to consumers through the presence temporarily of pathogenic forms. The direct application of germicidal agents to a drinking water, therefore, has always appealed to sanitarians as a prophylactic measure.

Historical.—Disinfection and even sterilization may, of course, be effected by boiling the water, but is virtually an impractical method on account of its cost where any considerable quantity of water must be treated. For this reason other methods of disinfection have been sought.

The application of chemical compounds to water and sewage for disinfecting purposes has received considerable study at different times during the past 50 or 60 years. In these investigations the halogens and their derivatives, compounds of copper, permanganates, caustic lime, and other agents capable of exerting an oxidizing action, have each and all been used from time to time on both water and sewage. It is only within the last decade, however, that the practice of disinfecting water supplies with chemicals has become quite general. The agent now commonly employed is chlorine, either in the gaseous form, or as calcium hypochlorite or bleaching powder.

For the disinfection and deodorization of sewage, English investigators employed calcium hypochlorite or bleaching powder as early as 1854. It was also recommended by the American Public Health Association in 1885 as the most efficient and cheapest disinfectant available. Certain German scientists also en-

dorsed its use. As early as 1892, the efficacy of the hypochlorites as water sterilizing agents was quite well known as the subject had received considerable laboratory study.

Bleaching powder was first applied to a public water supply in Maidstone, England, in 1897, following a typhoid fever epidemic. It was used to disinfect the distribution pipe lines of the city. Later in 1904 and 1905, it was again employed at Lincoln, England, for a like purpose. In 1896, James Hargreaves of Liverpool, England, in a paper read before the Liverpool Polytechnic Society, discussed the disinfection of sewage by means of chlorine manufactured by the electrolysis of salt. In 1906–07, Earle B. Phelps in this country experimented with chlorine and its compounds for the disinfection of sewage and water. In 1910, Major C. R. Darnall, U.S.A., made a series of experiments, using chlorine gas for the disinfection of drinking water. The development of apparatus for the practical application of chlorine gas has all taken place within the last 5 years.

The investigations of Johnson and Leal on the continuous application of small quantities of bleaching powder to the water supply of Jersey City in 1908, and its successful employment in the same year on the excessively polluted water of Bubbly Creek at the purification plant in the stock yards of Chicago, established its value in this country, and marks the beginning of its general adoption for the disinfection of public water supplies.

From a comparatively recent series of questions sent out by Mr. Francis F. Longley,[1] it appears that in the United States over 2,000,000,000 gal. of water per day are treated with either bleaching powder or chlorine gas. Four out of every five plants were using bleaching powder for disinfecting the water, while the remainder used liquid chlorine. In about 75 per cent. of the plants listed, the water supply was drawn from rivers, 20 per cent. from lakes, and the balance from the ground.

CHLORINE AND ITS COMPOUNDS

The cheapest and most easily obtained compound of chlorine is calcium hypochlorite or bleaching powder. As a byproduct of the soda-ash industry in Europe and of the electrolytic caustic-soda manufacturing process in this country, it may be prepared at a low cost. The use of bleaching powder in the

[1] Rep. Com. on Water Supplies. *Am. Jour. Public Health,* Dec. 4, 1914.

paper and textile industries creates a continuous demand for this chemical.

Preparation and Composition of Bleaching Powder.—Bleaching powder is prepared by passing chlorine gas over slaked lime. The latter absorbs and combines with the gas, forming a mixed salt in which calcium is united to two different acids. The commercial article is far from being pure as the following analyses will show:

	Per cent.
Calcium oxychloride,[1] $CaOCl_2$	64 93
Calcium chloride, $CaCl_2$	1 28
Calcium chlorate, $Ca(ClO_3)_2$	0.34
Calcium hydroxide, $Ca(OH)_2$	19.64
Calcium carbonate, $CaCO_3$	1.51
Calcium sulphate, $CaSO_4$	0 25
Oxides of Na, K, Mg, Al, Fe and Si	2 52
Moisture	9.95
	100.42

	Per cent. 1	Per cent. 2
Available chlorine[2]	37.00	38.30
Chlorine as chlorides	0.35	0.59
Chlorine as chlorates	0 25	0.08
Lime	44 49	43 34
Iron oxide	0.05	0.04
Magnesia	0.40	0.31
Alumina	0 43	0.41
Carbon dioxide	0 18	0.31
Silica, etc.	0.40	0.30
Water and loss	16.45	16.33
	100.00	100 00

It is evident that commercial bleaching powder is a mixture of chemical compounds, but that its chief active constituent is a mixed salt having the composition $CaOCl_2$. The activity of the compound is expressed in terms of the percentage of "available chlorine" that it contains, and which ranges from 30 to 37 per cent. The powder absorbs moisture gradually. The dry powder weighs about 49 lb. per cubic foot. It is not readily soluble in water (by weight 1 part in 20 of water), and solutions should not be made stronger than 0.5 to 1.0 per cent., if difficulties with

[1] CHARLES G. HYDE: "The Sterilization of Water Supplies by the Use of Hypochlorites." Pamphlet, 1911.

[2] LUNGE'S "Sulphuric Acid and Alkali." Vol. 3, p. 642.

sludge are to be avoided. Exposure to the air causes the compound to lose strength. Its offensive odor, corrosive properties and liability to deterioration on exposure, have been the chief objections to its use.

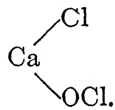
Sodium Hypochlorite.—By the electrolysis of a sodium chloride solution, sodium hypochlorite (NaOCl) is formed when the products of the electrolysis are not separated but are allowed to react upon each other. The hypochlorite solution thus produced is rather unstable and can not be transported without serious loss of strength. The cells in which electrolysis may be carried on are simple, and where the electric current can be obtained at a sufficiently low figure, the cost of the solution is not high. The control of the preparation of the solution, however, is not as easy as might at first appear. Inability to store the solution, on account of its unstable character, and its higher cost as compared to bleaching powder, have prevented its coming into practical use in spite of the considerable amount of experimental work done to develop it for water-sterilization purposes.

Liquid Chlorine.—The chlorine gas produced by the electrolysis of sodium chloride in the manufacture of caustic soda is used in large quantities to produce bleaching powder, as heretofore stated. Of late years, however, an outlet for more or less of this gas in the liquefied form has been found. The purified gas is dried and liquefied by pressure, and is put on the market in steel drums or cylinders holding from 100 to 140 lb. each. The chlorine is very pure, containing no impurities except traces of oxygen, carbon dioxide and nitrogen. It is free from moisture. The pressure in the drums varies with the temperature, and ranges from 54 lb. per square inch at 32°F., to 216 lb. at 122°F. Chlorine gas is dangerous to inhale because of its corrosive action on the mucous membrane of the throat and lungs. Great care should be exercised in dealing with it, and especially when it is under pressure. Leaky fittings or apparatus should be repaired immediately. Storage of cylinders containing the gas under pressure should be kept in as cool a place as possible. Supply tanks and apparatus for measuring and conveying the gas should be placed, if possible, in rooms maintained at a temperature of not much less than 50°F., since the conversion of the liquid chlorine to the gaseous form in the supply tanks requires heat, which is, of course, drawn from the metal container in which the liquid stands. The more rapid the evaporation of the liquid

chlorine, the greater the lowering of its temperature, and consequently the lower the initial pressure of the gas being formed. This is of practical importance in adjusting pressure reducing valves in order to maintain a uniform flow of the gas.

CHEMISTRY OF THE ACTION OF CHLORINE AND ITS COMPOUNDS IN WATER

Bleaching powder may be regarded as a calcium-chloro-hypochlorite, or better as a mixed salt in which the two acids, hydrochloric and hypochlorous, have the common base calcium to neutralize them. This may be graphically shown as follows:



When this compound is placed in solution in water, it undergoes a molecular rearrangement, and appears to become a true hypochlorite, plus calcium chloride ($2CaOCl_2 = Ca(OCl)_2 + CaCl_2$). It has been found that aqueous solutions of bleaching powder show the presence of Cl, ClO and Ca ions.

When a bleaching powder solution is applied to a natural water containing the bicarbonates of calcium and magnesium, the following reaction takes place:

$$Ca(OCl)_2 + CaCO_3 + H_2CO_3 \rightleftarrows 2CaCO_3 + 2HOCl.$$

The hypochlorous acid being a weak acid has no appreciable effect on the calcium carbonate formed. This acid, however, will decompose into hydrochloric acid and oxygen with liberation of a considerable amount of energy in the form of heat. This occurs whether the acid is present alone in the water, or with other substances. If some oxidizable substance is present, the decomposition of the hypochlorous acid is rapid, and the energy of the reaction accounts sufficiently for the destructive effect produced on living cells like those of the microörganisms.

If chlorine gas is applied directly to a water, the following reaction occurs:

$$Cl_2 + H_2O \rightleftarrows HCl + HOCl.$$

This reaction being strongly reversible permits of the formation of but small quantities of HOCl at any time, and requires the removal of one or both of the products of the reaction in order to bring it to completion. This occurs in two ways in a natural

water which is slightly alkaline from the presence of calcium and magnesium carbonates, and contains oxidizable organic matter as well. The HCl reacts with the carbonates to form calcium chloride and carbonic acid. The HOCl is decomposed into HCl and oxygen, which latter acts upon any oxidizable matter that may be present.

(*a*) $CaCO_3 + 2HCl \rightarrow CaCl_2 + H_2CO_3$
(*b*) $2HOCl \rightarrow 2HCl + O_2$.

Equation (*b*) represents the liberation of oxygen from hypochlorous acid, which latter is formed when either a hypochlorite or chlorine is dissolved in water. The energy liberated by the decomposition of the hypochlorous acid, as previously stated, explains the powerful oxidizing action of the evolved oxygen, and the destructive effect upon the microörganisms. Chlorine or the hypochlorites are, therefore, merely agents for the production of oxygen under conditions which render it extremely active.

The term "available chlorine" in hypochlorites refers to the oxygen-releasing value of the chlorine in these compounds. From an examination of the reaction between chlorine gas and water, it will be noted that 2 atoms of chlorine are required to decompose 1 molecule of water and set free its atom of oxygen; or in other words, that 16 parts by weight of oxygen have been liberated by 70.9 parts of chlorine. This is in the ratio of 1.00 part by weight of oxygen to 4.43 parts by weight of chlorine, It will also be seen that pure hypochlorous acid or a pure hypochlorite, when decomposed, liberate but 1 atom of oxygen for each atom of chlorine, or double the quantity liberated when pure chlorine reacts with water. Solutions of the hypochlorites are, however, always mixtures, containing 1 molecule of the chloride for each molecule of the hypochlorite; viz., $CaCl_2$, $Ca(ClO)_2$ or NaCl, NaClO. The active or oxygen-releasing chlorine in the hypochlorites, therefore, bears the same relation to the total chlorine that it does in solutions of the pure element in water.

The "potential oxygen," as it is sometimes called, which is produced by the decomposition of bleaching powder, forms from 7 to 8 per cent. of the weight of the latter. On the other hand, where pure chlorine gas is used, it produces theoretically 22.5 per cent. of its weight of "potential oxygen." Various losses in actual practice lower the proportion of "potential oxygen" produced in using either the hypochlorites or pure chlorine gas.

Application of Chlorine and Its Compounds.—The general methods employed for the preparation and application of bleaching-powder solutions are described in more or less detail in Chapters XXI and XXII. As will be noted in reading these chapters, aside from providing for the corrosive properties of the solution, and for its offensive odor, the tanks and orifice apparatus may be similar to or the same as those used for solutions of other chemicals that are commonly prepared and applied to water in purification processes.

Sodium Hypochlorite Apparatus.—The probability that sodium hypochlorite will ever come into use as a disinfecting agent for

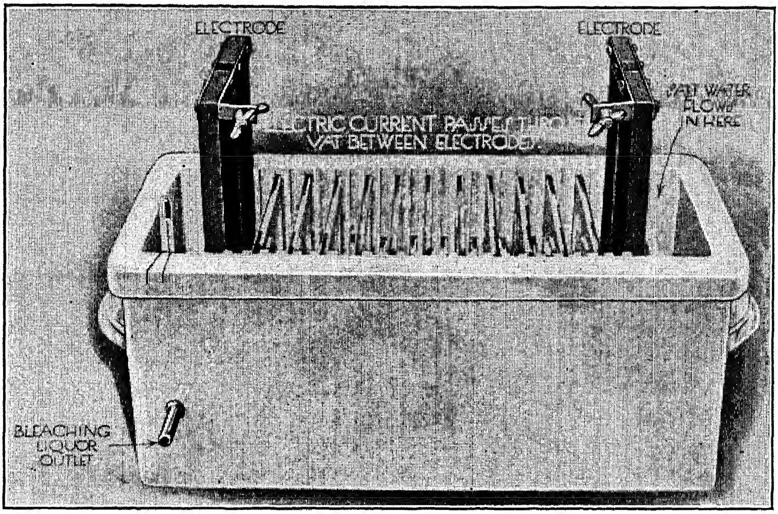


FIG. 118.—Cell for electrolyzing a common salt solution.

public water supplies is slight, in spite of the considerable experimental work which has been carried on to determine the best methods for controlling its production, storage and application. It has one marked advantage over the use of bleaching powder in not having to dispose of an offensive sludge. Its preparation is relatively simple, but the control of the strength of the solution is not particularly easy, nor can any storage of the liquid be made, except for relatively short periods, without a considerable decrease in strength.

The apparatus for preparing sodium hypochlorite consists of a tank for holding a solution of common salt, cells for the electrolysis of the salt solution, and the necessary apparatus for

applying the electric current. An ordinary orifice box to control the quantity of the solution to be applied to the water is also necessary.

The electrolytic cell may be made of soapstone or procelain. Carbon (graphite) electrodes are used. One cell now on the market consists of a small soapstone tank filled with a series of graphite electrodes which are separated by glass baffles. The salt solution enters a separate compartment at one end of the cell, and then passes up and down between the faces of the carbons, and out at the opposite end of the cell. The electric current enters the large positive electrode or anode at one end and passes out at the corresponding negative electrode or cathode at the opposite end. The opposite faces of the intermediate carbons act as positive and negative electrodes, and thus really form a number of cells in series. The following reaction occurs as the current passes through the salt solution:

$$NaCl + H_2O \rightleftarrows Cl + NaOH + H.$$

The chlorine is liberated at the anode and the hydrogen at the cathode. Around the cathode is also produced a solution of caustic soda. The latter is the result of the action of the sodium on the water molecule, viz., $Na + H_2O \rightarrow NaOH + H$. The chlorine also reacts with a molecule of water to form hypochlorous acid (see page 373), which in turn reacts with the caustic soda to form sodium hypochlorite.

$$NaOH + HOCl \rightleftarrows NaOCl + H_2O.$$

The continuous flow of the brine solution through the cell makes possible the interaction of the products of the electrolysis. Other secondary reactions occur, such as the formation of sodium chlorate, and the reduction to the chloride of a portion of the hypochlorite and chlorate first formed. In order to minimize the effect of these secondary reactions, it is necessary to keep the brine solution below a temperature of 100°F.

Theoretically 1 amp. of current flowing for 1 hr. will produce 1.322 grams of chlorine from 2.182 grams of sodium chloride. On account of the low efficiency of cells of the type described above, however, it has been found in practice that it required from 4 to 5 amp. of current and about 8 lb. of salt to produce 1 lb. of chlorine. Experiments made by Mr. G. A. Johnson indicated that with a consumption of electrical energy equal to 3.2 kw.-hr., 1 lb. of chlorine could be produced from 8.3 lb. of salt at

a cost of 6.46 cts. per pound, where the current cost 1.5 cts. per kilowatt-hour, and the salt $4 per ton.

From experiments made in the laboratories of the Ohio State Board of Health with cells for electrolyzing a sodium chloride solution, the following deductions were made and published in its *Quarterly Bulletin*, vol. 1, No. 4, 1909.

"1. The cost of production of chlorine electrolytically depends on the cost of the salt and the cost of the electricity.

"2. Since the cost of the current is based on kilowatts or the product of voltage times amperage, and since the voltage of the cell remains practically constant, it follows that to obtain a low cost of operation the amperage must be kept down.

"3. Amperage can be lowered by increasing the density of the current, by lowering the temperature of the cell, and by an increased rate of flow. A limit is placed on the first and third factors by the cost of the salt, and again on the third factor by the capacity of the cell. In the practical working of the cell it would be important to have a supply of cold water at little or no cost for cooling purposes.

"4. The highest efficiency at ordinary temperatures was obtained with a concentration of brine of about 40,000 parts per million with a rate of flow of about 1,300 c.c. per minute.

"5. At an artificially lowered temperature, the best results were obtained at 46°F. with a concentration of 50,000 parts per million and a rate of flow of 1,680 c.c. per minute.

"6. Hot weather conditions show the cell at its lowest efficiency.

"7. No attempt was made to determine the point of the highest efficiency of the cell or to determine the amount of chlorates and side reactions formed since it became evident that the cell could not be run under practical conditions at a figure that could compete with chloride of lime."

Liquid Chlorine Apparatus.—There have been several devices placed upon the market for applying chlorine gas obtained from liquid chlorine. The apparatus now in successful operation has been the outcome of considerable experimental study, and required the overcoming of difficulties arising from the pressure variations of the gas at different temperatures, and from its extremely corrosive effect on metals wherever it came into contact with the latter in the presence of moisture. The slight solubility of the gas in water has also required the use of special diffusion devices, or of absorption apparatus.

The principle involved in the construction of most of the apparatus now being used for the application of chlorine gas is that of

maintaining by means of suitable pressure reducing valves a constant drop in pressure across some form of orifice. An apparatus of this type now on the market[1] consists essentially of a reducing pressure valve in the pipe leading from the supply tank, a second regulating valve following the first valve, an orifice plate and an absorption tank. The first valve reduces the initial pressure of the gas coming from the supply tank to about 15 lb. per square inch. The gas then passes through the second adjustable reducing pressure valve by which any desired pressure may be maintained over the orifice plate in the pipe line. Between the second regulating valve and the orifice plate is a branch pipe to which a chlorine pressure gage is attached. This gage is calibrated in pounds per hour, and thus serves to indicate the rate of discharge of the gas for any given setting of the second regulating valve.

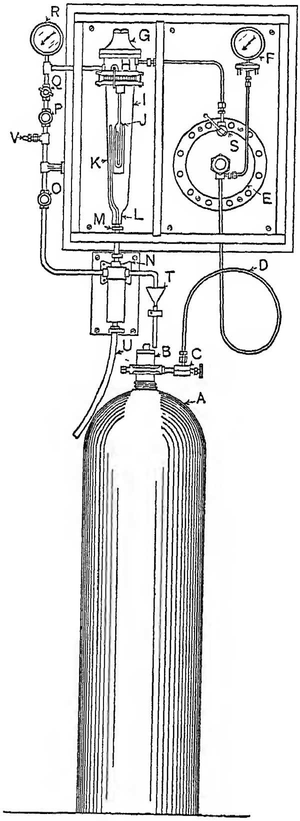


FIG. 119.—Manual control chlorinator, direct-feed type. (*Wallace & Tiernan Co.*)

After passing the orifice plate, the gas is conducted to the bottom of an absorption tower. Water admitted at the top of the tower flows down over broken material, which must be of such a nature as to be unaffected by the chlorine and water, and which offers a large surface for the absorption of the ascending gas by the descending water. The chlorine solution thus produced is conducted to the water to be treated.

Another apparatus,[2] essentially along the same lines as that just described, consists of a gage to show the pressure in the supply tank, a pressure compensating valve for maintaining a constant drop in pressure across a control valve, a control valve, a chlorine gas flow meter which has been calibrated empirically,

[1] Made by the Electro-Bleaching Gas Co., New York, N. Y.

[2] Manufactured by the Wallace & Tiernan Co., Inc., New York, N. Y.

a visible glass orifice through which the chlorine is measured, a check valve, a back pressure gage which will indicate any stoppage of the flow of gas, and a chlorine absorption chamber. In some types of apparatus the absorption chamber is not used as the solution of the gas is not attempted in the device itself, but is fed directly through a "carborundum diffusor" placed in the water to be treated. The "diffusor" becomes saturated with water due to the capillary effect of the minute passages within

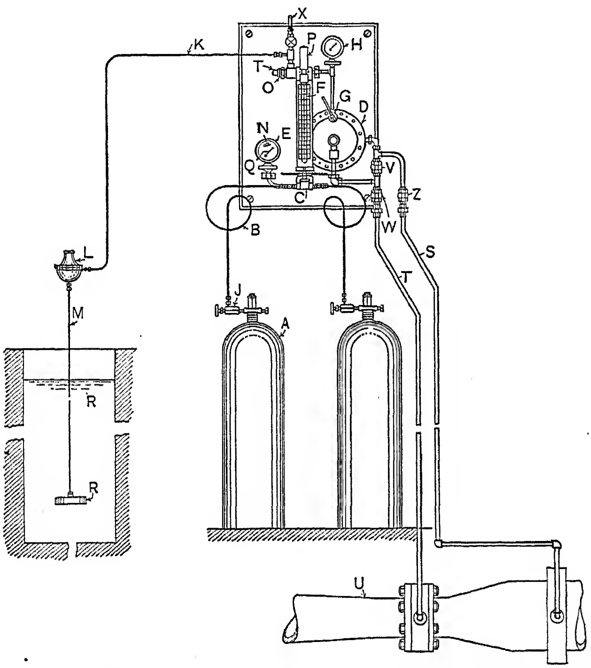


FIG. 120.—Automatic control chlorinator, direct-feed type. (*Wallace & Tiernan Co.*)

the carborundum disk. The pressure of the chlorine entering the center of the "diffusor" enables the gas to force its way through these small passages and to become saturated with moisture. On emerging from the "diffusor" in minute bubbles, it is readily dissolved by the water surrounding the carborundum disk.

For feeding very small quantities of chlorine gas, a small volumetric pulsating glass meter has been designed, which utilizes the pressure of the entering gas to displace a definite volume of

water in a siphon connected directly with the absorption chamber of the apparatus.

All of the types of apparatus described above are provided with flexible tube connections leading to the supply tanks, since

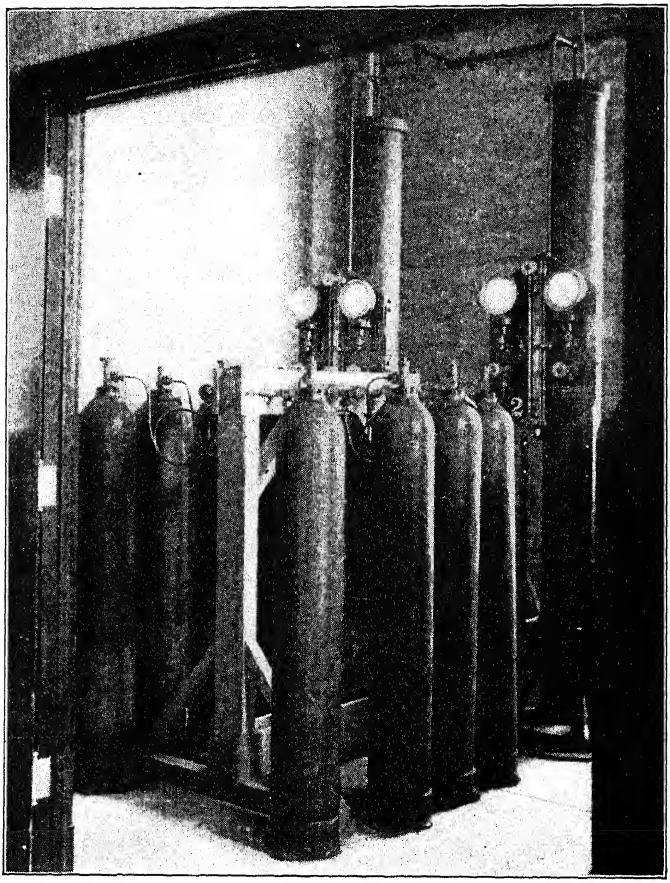


FIG. 121.—Liquid chlorine apparatus in head house of St. Louis filter plant. (*Electro-Bleaching Gas Co.*)

the latter are usually placed upon platform scales, whereby the loss of weight of the gas from the cylinders may furnish a check upon the quantity of gas passing through the apparatus. Other attachments to these devices enable them to operate automatically in unison with and proportional to variations in the rate of flow of the water being treated, through connections to floats or Venturi meters.

In another chlorine apparatus which was invented for the disinfection of water and sewage, a different principle from that employed in the previously described devices, was utilized. In this device the action depends upon the loss of weight of the gas from a supply tank, instead of upon the maintenance of a constant drop in pressure across an orifice.

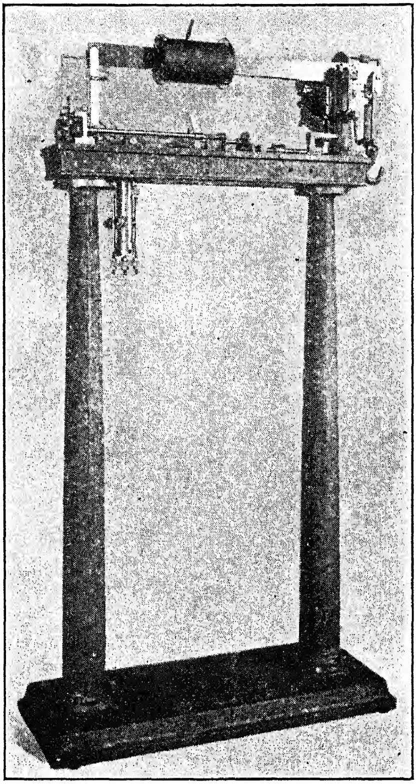


FIG. 122.—The Leavitt-Jackson chlorine apparatus.

The liquid chlorine to be applied is stored in steel flasks or cylinders containing about 100 lb. This flask or tank is suspended on a hook from a sensitive scale beam, mounted on hardened steel knife edges. The counterbalancing weight on the beam can be fed forward either at a constant rate, or in proportion to the rate of flow of the water being treated. The beam communicates through a system of levers with a controlling valve connected by rubber tubing to the suspended cylinder.

Any change in equilibrium of the system, due to the weight being set forward along the beam, causes the valve to open or close automatically to the proper size of orifice. This allows the exact weight of gas to escape continuously from the tank, and keeps the scale beam in balance. This apparatus is known as the Leavitt-Jackson chlorinator.

For conveying dry chlorine gas iron or steel pipe may be used. Wherever chlorine and water in any form are in contact with iron, brass, copper, tin or any metal attacked by hydrochloric acid, corrosion will take place. Silver tubing is used for conveying the gas into water, as the silver chloride first formed on the tube acts as a protective coating to the metal underneath it. Glass, hard rubber and stoneware are also commonly employed for delivering the gas or its aqueous solution to the water to be treated. The author has used ordinary garden hose for conveying the gas directly into the water with considerable success.

OZONE

Ozone is a gas containing three atoms of oxygen. It is much more soluble in water than is oxygen, since at 12°C. 100 volumes of water will dissolve 50 volumes of ozone at atmospheric pressure. Ozone is relatively stable only when mixed with much oxygen. When dry, cold, oxygen gas is subjected to the action of electric waves, only about 7.5 per cent. of the gas is converted into ozone. Ozone is a more active oxidizing agent than oxygen, and like the hypochlorites its activity is due to the fact that it contains much more energy than oxygen.

Formation of Ozone.—A silent electric discharge passed through air or oxygen gas will produce ozone. A spark or arc discharge of electricity passed through air, however, produces oxides of nitrogen, but little or no ozone. The necessity for producing a silent discharge of the electric current in the formation of ozone was noted by the earliest investigators, but the precise reason for it is not well understood even today. Commercial ozonizers are operated by high-tension alternating currents, and the chief feature of their design is the care taken to avoid any spark or arc discharge.

As early as 1875 Werner von Siemens invented an ozonizer consisting of two concentric glass tubes, one slightly larger than the other, coated with tin foil on the external and internal faces,

respectively. The two coatings were connected with two poles of a high-potential electrical machine, and a current of dry air was passed along the annular space between the two glass tubes. Upon the discharge of the current between the poles during the passage of the air, ozone was produced, the amount formed being dependent on the rapidity with which the air was withdrawn, whereby heating was avoided, and upon the absence of sparking.

Modern Ozonizers.—Practically all of the modern apparatus for the commercial production of ozone is based upon the general principles developed in Siemens' original experiments. The Siemens and Halske (Fig. 123) ozonizers are of the tube type, in which air is drawn along an annular space in which high-tension

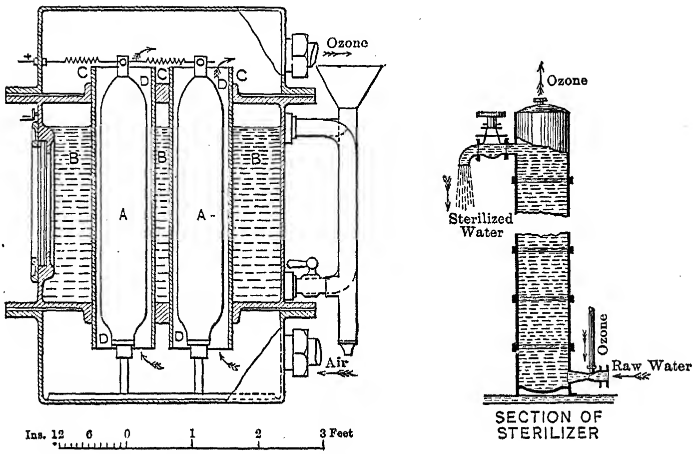


FIG. 123.—Siemens-Halske ozone apparatus.

electrical discharges occur. One glass tube has, however, been dispensed with, and an internal metal cylinder, separated by a slightly larger glass tube from the external cylinder of tin foil, forms the ozonizing unit in each apparatus. The outer cylinder of the apparatus is surrounded with water in order to keep down the temperature, and all the external parts are kept at the earth potential to reduce risks from shocks from the high-voltage currents employed. The Otto (Fig. 125) ozonizer of the latest type consists of a series of glass plates, coated on alternate sides with tin foil, and separated by narrow strips of insulating material, which allow air spaces between adjacent sheets of the glass.

The Rosenberg ozonizer differs from the two preceding forms

of apparatus in that glass is replaced by micanite, and the continuous sheets of tin foil, which form the electrodes in other apparatus, are replaced by copper or by an aluminum-alloy gauze of 40

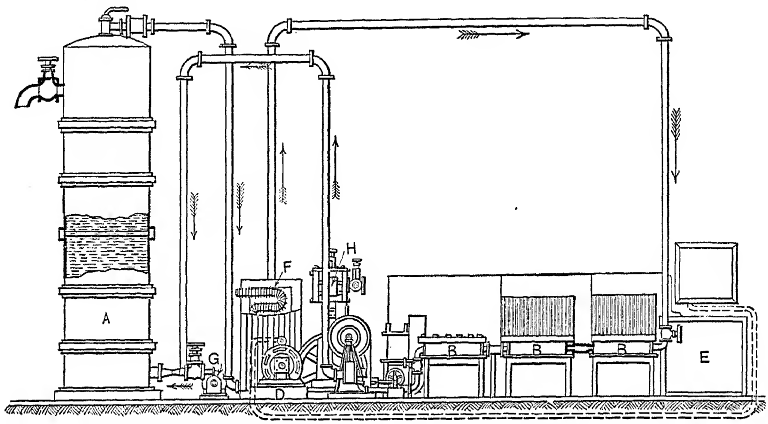


FIG. 124.—Vosmaer system for ozonizing water.

meshes to the inch. Current at 4,500 volts is said to pass between the electrodes in this apparatus without sparking. De-Fries' ozonizer is similar to Rosenberg's, and consists of a hori-

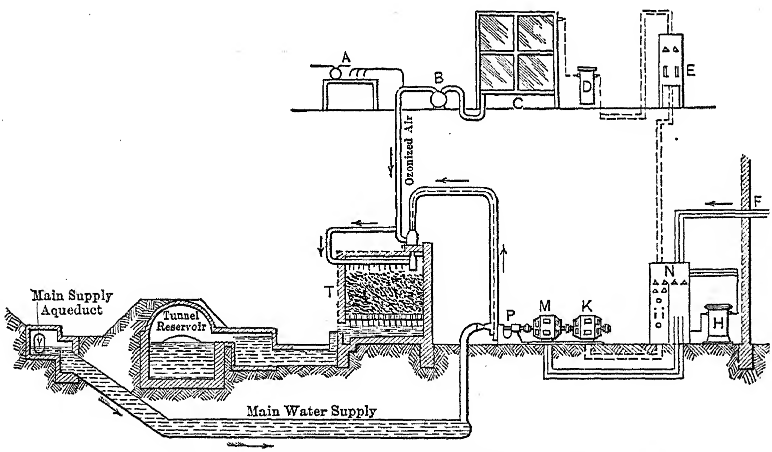


FIG. 125.—Otto system for ozonizing water.

zontal brass trough fitted with a plate-glass cover from which are suspended half-disks of brass with serrated edges. The brass trough is water-cooled. A silent discharge results between the

edges of the disks and the surface of the trough, when a current of 2,000 volts is passed through the apparatus. The air drawn through the apparatus is ozonized as it passes between the electrodes while the electric current is discharging.

Ozonizing apparatus for the disinfection of water has been used in America in a few places, the best known plants being those installed at Ann Arbor, Mich., Lindsay, Canada, and at the

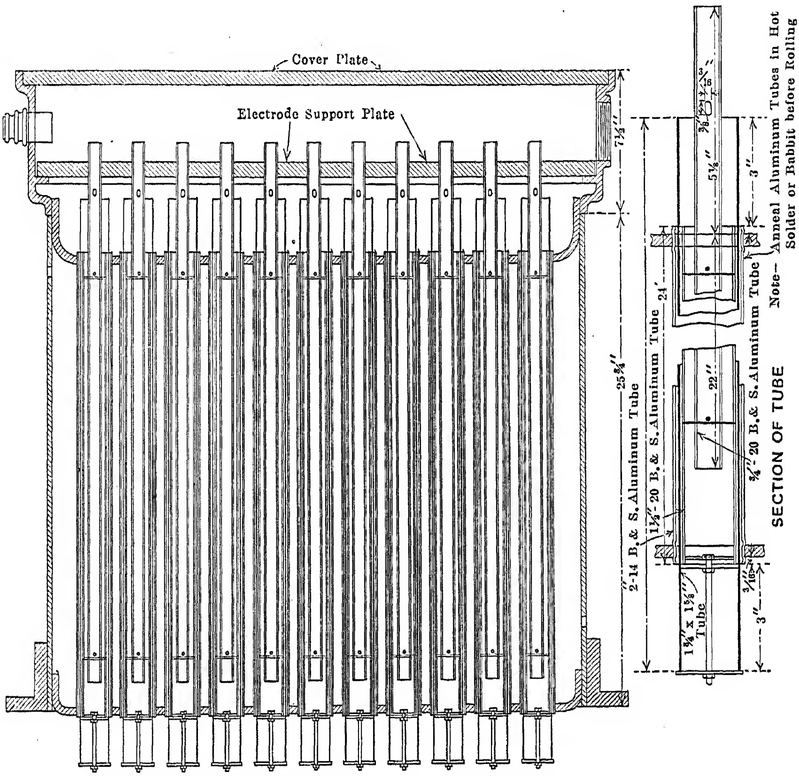


Fig. 126.—Details of ozone generator.

Herring Run supply of the Baltimore County water system (Fig. 126). The company owning the latter water supply system has used the Bridge type of ozone generator consisting of cylindrical aluminum electrodes and micanite dielectrics.

Yield of Ozonizers.—In order to obtain the best yield (Fig. 127) from an ozonizer, the entering air must be dry. This may be effected by refrigeration, and apparatus for this purpose

may, therefore, be considered as an essential part of the ozone plant.

The Siemens-Halske ozonizers of the type shown in the accompanying cut requires for its operation 1 hp. per hour, and produces 13.5 to 27 grams of ozone per hour according to the amount of air

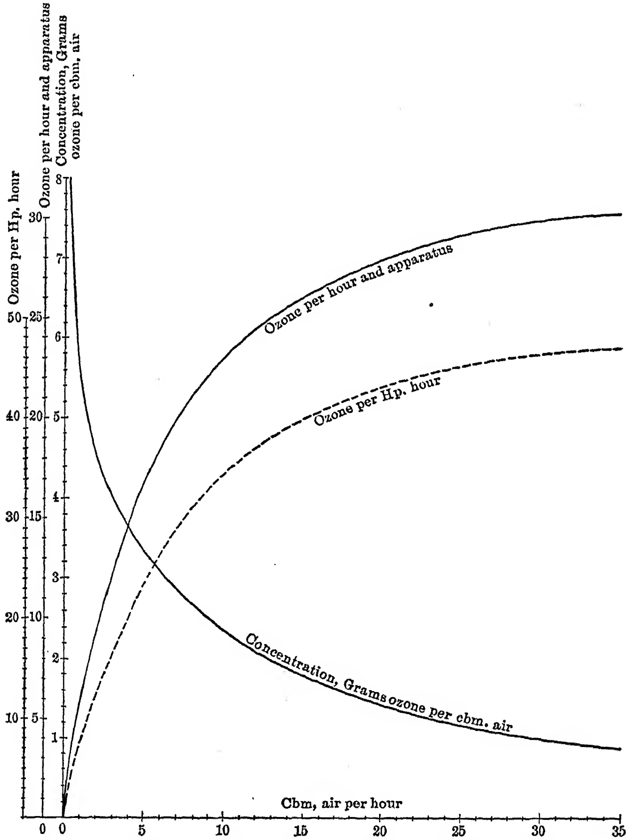


FIG. 127.—Curves of yield of an ozone apparatus.

passed through it, and the degree to which it has been previously dried. A Gerard ozonizer (10-tube unit) used at Great Falls, S. C.[1] is capable of producing about 20 grams of ozone per hour. By actual test these units required about 400 watts each, when the air passed through the ozonizer at the rate of 2 cu. ft. per minute.

[1] *Eng. Record,* vol. 64, July 1, 1911.

Application of Ozone.—The ozonized air produced by the generators is usually applied to the water to be treated by means of scrubbers. These are towers from 12 to 15 ft. in height, in which the water entering the top of the tower flows down through a bed of coarse gravel. The ozonized air is blown in at the bottom of the tower, and as it ascends comes into contact with thin films of the water trickling down through the gravel bed. The ozone which is absorbed exerts its oxidizing action upon the organic matter that is dissolved and suspended in the water, including, of course, the bacteria. The longer the contact which the water has with the ozone, the greater the purification effected. Any excess of ozonized air may be returned to the ozonizers again if desired. The purified water is drawn off at the bottom of the towers.

In some cases the ozonized air is merely pumped into the bottom of a column of water, and allowed to rise through it. This method does not produce as complete a contact between the water and the ozone as do the scrubbing towers, and, beside, requires more power to overcome the pressure of the water column.

At the Herring Run plant of the Baltimore County Water and Electric Co., the water flows to the ozone plant directly from the reservoirs, and is treated with ozone by means of aspiration. The falling water sucks the ozonized air directly from the generators, and then passes through a mixing chamber to the open suction well of the pump. Any excess of ozonized air escapes in the open suction well.

The largest ozone plant that has probably ever been installed is at Petrograd, Russia. This plant contains 128 Siemens and Halske ozonizers of the tube type, and five sterilizing towers. A refrigeration plant dries the air before it is sent to the ozonizers. Injectors operating at a water pressure of 14 ft. suck ozonized air from the ozonizers, and drive it mixed with water to the sterilization towers. Absorption of the ozone by the water takes place in the injectors at the top of the sterilization towers, and in the passage of the water downward through the towers, where it comes into contact with ozonized air rising from the bottom of the towers. The ozonized water flows over cascades to remove the excess of air before entering a storage tank.

The ozonizers are supplied with a current having a voltage of 7,000. The volume of water treated is about 13,000,000 gal. per day.

Quantity of Ozone Used.—The amount of ozone required to properly disinfect different waters must of necessity vary considerably. The extent of the original pollution of the water, and the degree of purification effected prior to treatment with ozone, will naturally affect the amounts required. The best results, so far as the destruction of the bacteria are concerned, will be obtained with those waters which contain the least amount of dissolved and suspended oxidizable matter. Turbid waters should, therefore, be settled and filtered before attempting disinfection. Those waters containing vegetable coloring matter in colloidal solution will require more ozone than those free from it.

At certain German plants[1] the average consumption of ozone for fairly clear waters was found to be 1.3 grams per cubic meter,

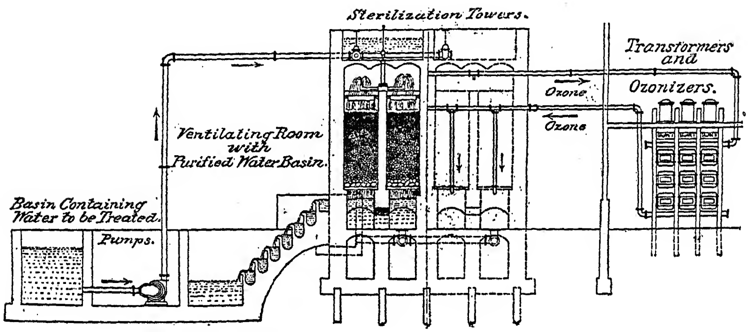


Fig. 128.—The Paderborn, Germany, water ozonizing plant.

equivalent to about 5,000 grams per million gallons. Probably from 3,000 to 8,000 grams of ozone per million gallons will be sufficient for disinfecting most waters, if effective methods of application are used.

Extent of Use of Ozone.—In America ozone is not used for disinfecting water to any great extent. In Europe, however, there are several water-works plants using this agent. The largest plant is at Petrograd, Russia, as previously noted. Plants treating over 1,000,000 gal. of water per day may be found also at Wiesbaden, Germany; Florence, Italy, and at Chartres, France. At Nice and St. Maur, in France, plants handling 10,000,000 and 6,000,000 gal. of water per day, respectively, are sterilizing with this agent. A number of smaller plants are also to be found in different parts of Europe.

[1] Paderborn and Wiesbaden, Germany.

ULTRA-VIOLET LIGHT

The favorable effect of light upon most forms of life is common knowledge. On the other hand, the destructive action of sunlight upon many of the lower forms of plant life is also well known. The harmful effects of artificial light rays produced by the electric arc under certain conditions have received considerable attention by scientists during the last decade. The results of these studies indicate that the shorter wave lengths of light exert a marked destructive action upon bacterial life.

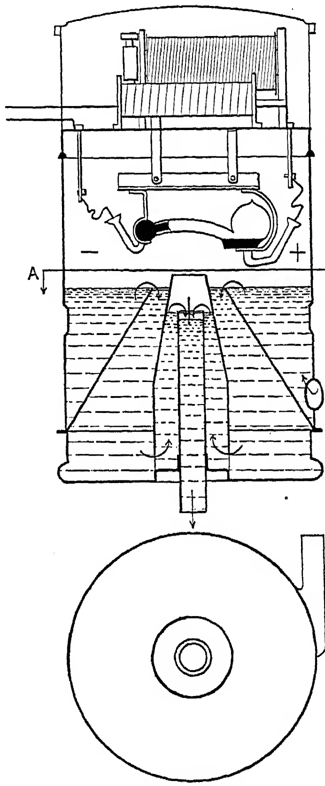


FIG. 129.—Ultra-violet ray sterilizer for water.

Historical.—By exposing bacterial cultures to the light rays from different portions of the spectrum, or by using filters and screens which will only permit rays of certain wave lengths to pass, it is possible to note directly the germicidal effects produced by light rays of different wave lengths. For example, it has been found that colonies of bacteria growing upon an agar plate were not affected by the red rays of the spectrum, but that growth was prevented when exposed to the violet rays.

The experimental work of Henri, Helbronner and von Recklinghausen, at the Sorbonne University in Paris, demonstrated quite conclusively that the shorter the wave length of light the greater its abiotic power. The shorter wave lengths of light are emitted at the violet end of the spectrum. The invisible rays of light beyond the violet rays have the shortest wave lengths and are the most effective bactericidal agents. Rays having a wave length of 0.0003 mm. and less were found to destroy bacteria in a comparatively short space of time. Many of the pathogenic forms can be destroyed by an exposure of 10 to 40 sec.

Even Paramecia and yeast cells were destroyed by exposures of 3 to 5 min. Exposures of even 1/10 sec. are frequently sufficient to kill some bacteria.

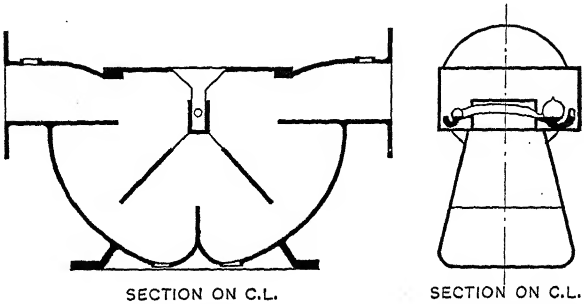


Fig. 130.—Sectional view of ultra-violet ray water sterilized.

Production of Ultra-violet Light.—The mercury vapor quartz lamp has been found to emit light rich in ultra-violet rays, and

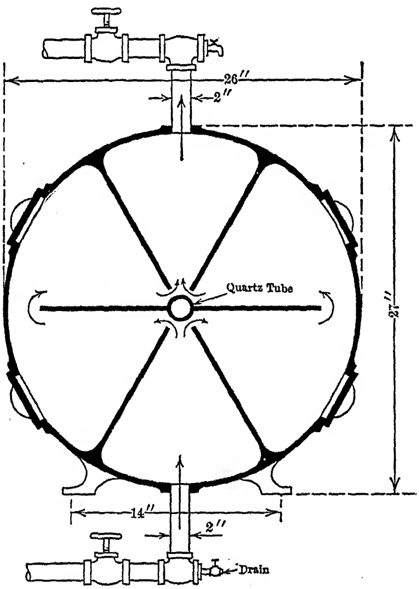


Fig. 131.—Ultra-violet sterilizer for use with water under pressure.

the conditions under which the largest number of these rays were produced have been carefully investigated by von Recklinghausen and his associates. This lamp, which consists of a fused rock-

crystal (quartz) tube containing mercury, is the only practical source of ultra-violet light that up to the present time has been adapted to the conditions required for the sterilization of water.

In lamps of this type metallic mercury forms the cathode and the arc passes through rarified mercury vapor. The light produced is not due to the temperature of the arc, but to the luminescence of the vapor. The lamps may be operated on the usual distribution voltages for direct currents. An alternating current must be first rectified in the usual way before being sent to the lamps.

The Pistol Lamp of Max von Recklinghausen.—A recent improvement in lamps for sterilization purposes has been invented by Dr. Max von Recklinghausen. This is known as the "pistol lamp" because of its shape. It operates on a 500-volt current and requires 3 amp. Its luminous tube is U-shaped, the two branches of the U being very close together. The luminous portion of the lamp is enclosed in a 2-in. quartz tube, which forms the lamp chamber, and which prevents the water coming into contact with the lamp proper. By making the quartz tube U-shaped instead of straight, it has been possible to utilize all of the light emitted instead of only 60 per cent. of it as in former lamps.

The efficiency of this lamp as compared with those first produced has been increased not only on account of its shape, but also because of its greater production of ultra-violet light. About 10 times as much ultra-violet light is produced in the pistol lamp as in the old 220-volt lamp, although the voltage is only doubled. Compared with the 3.5 amp. lamps, the ultra-violet rays are 50 times as powerful, although the wattage is only four times as great.

Conditions Necessary for Efficient Lamp Operation.—The hotter the mercury quartz lamp becomes, the more ultra-violet light it will produce. By properly cooling the electrodes and by having large electrode containers, it has been found possible to control the temperature of the lamp, if the tubes are of the correct dimensions and a proper amount of resistance is inserted. The heat must not be so great as to cause the quartz tube to disintegrate and become opaque. Under normal conditions the luminous tube of the lamp has a temperature of about 800°C. A temperature of 700°C. has been successfully maintained for many thousand hours. As soon as a lamp is worn out, it frequently extinguishes itself, and must be started again. This should serve

as a warning to those in charge that a renewal of the lamp is needed.

Since close and direct contact of the bacteria with the ultra-violet rays is productive of the best results, efforts have been made in designing the apparatus to so arrange it that the water shall flow slowly through the illuminated zone. In some apparatus, where the lamp is suspended immediately above the water, the latter is made to pass twice through the light zone by means

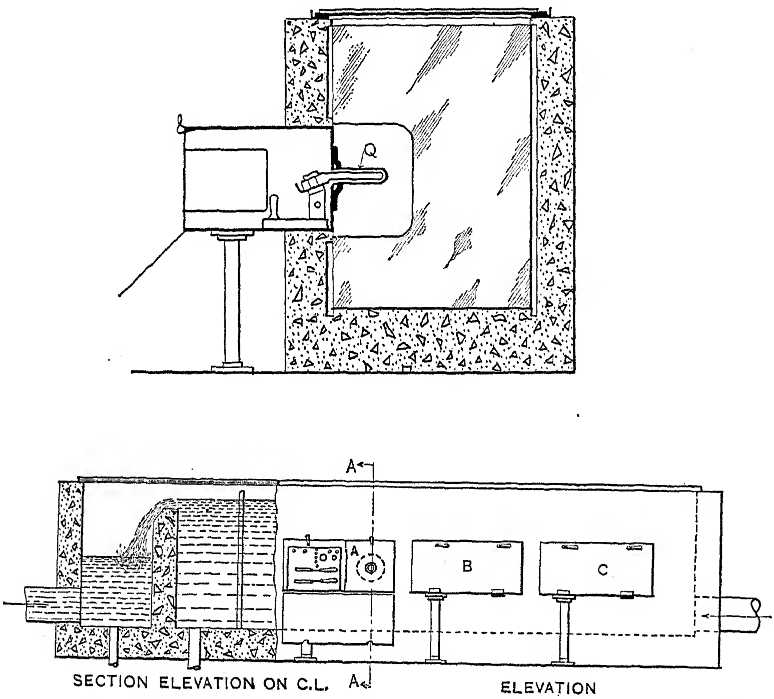


FIG. 132.—Multiple lamp flume for ultra-violet ray sterilization of water.

of baffles. A better arrangement, especially where large volumes of water are to be treated, is to provide a flume through which the water flows. The lamps are placed in the sides of the flume, and of course must be made accessible for purposes of adjustment and renewal. Baffles also are used in order that a mixing action may be produced, thereby lessening the chances for any of the bacteria escaping a sufficient exposure to the light rays.

Since actual exposure to the light rays is absolutely necessary, if the microörganisms are to be killed, the presence of any con-

siderable quantity of suspended matter or of colloidal vegetable coloring matter in the water, which will screen or diminish the intensity of the light rays, can not be permitted. The presence of as much as 20 parts per million of suspended matter in a water, or of 40 parts of color, have not prevented successful sterilization according to Dr. von Recklinghausen. However, it is preferable to have the water as clear and colorless as possible, before attempting to sterilize it by this method.

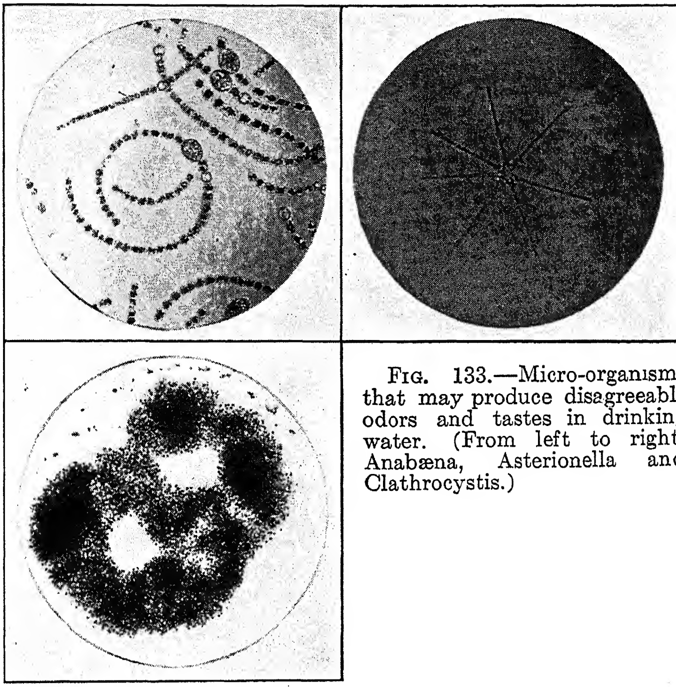


FIG. 133.—Micro-organisms that may produce disagreeable odors and tastes in drinking water. (From left to right, Anabæna, Asterionella and Clathrocystis.)

Algicides.—The pollution of public water supplies by growths of algæ, diatoms and other microörganisms is not uncommon, and, at those periods when the growths are particularly abundant, may require the use of remedial measures. These growths may become extremely troublesome in the operation of filters by clogging the filtering medium.

By decomposing on the surface of the filter bed these organisms may also produce obnoxious odors and tastes in the filtered water. The algæ, and especially the blue-green algæ (Cyanophyceæ), are more frequently the cause of bad odors and tastes,

although the diatoms and the infusoria are both sometimes found to be the source of these troubles. Certain of the tastes and odors are characteristic of the growth of these organisms, while others, which are usually extremely disagreeable, are the results of the decomposition of the organic matter after the death of the organisms.

The following table taken from a paper by D. D. Jackson and the author "On Odors and Tastes of Surface Waters with Special Reference to Anabaena" (*Technology Quarterly*, vol. 10, December, 1897), describes some of the characteristic odors of growth and decay.

Microörganisms	Natural odor	Odor of decay
DIATOMACEÆ:		
Asterionella	Aromatic to fishy.	
Tabellaria	Aromatic.	
Meridion	Aromatic.	
CYANOPHYCEÆ (Blue-green Algæ):		
Anabaena	Mouldy, grassy.	Pig pen.
Rivularia	Mouldy, grassy.	Pig pen.
Clathrocystis	Sweet, grassy.	Pig pen.
Coelosphærium	Sweet, grassy.	Pig pen.
Aphanizomenon	Faintly grassy.	Pig pen.
CHLOROPHYCEÆ:		
Volvox	Fishy.	
Eudorina	Faintly fishy.	
Pandorina	Faintly fishy.	
INFUSORIA:		
Uroglena	Fishy and oily.	
Synura	Ripe cucumbers.	
Dinobryon	Fishy, like rockweed.	
Bursaria	Irish moss or salt marsh.	
Peridinium	Fishy, like clam shells.	
Cryptomonas	Candied violets.	
Mallomonas	Faintly fishy.	

The research work of George T. Moore and Karl F. Kellerman, of the Bureau of Plant Industry of the U. S. Department of Agriculture, called attention some years ago to the value of copper sulphate in preventing and in destroying growths of microörganisms in water. Its value for this purpose has been amply demonstrated in practice, and is now a recognized method for

rectifying difficulties of this character. The following tables, compiled by Dr. Karl F. Kellerman,[1] showing the frequency of occurrence of various genera of microörganisms in public water supplies, and the amounts of copper sulphate necessary to kill these organisms, are of interest in this connection.

TABLE SHOWING THE OCCURRENCE OF GENERA OF MICROÖRGANISM MOST FREQUENTLY REPORTED AS CAUSING TROUBLE IN RESERVOIRS AND PONDS

	Anabaena	Beggiatoa	Asterionella	Chara	Cladophora	Clathrocystis	Conferva	Crenothrix	Fragillaria	Navicula	Oscillatoria	Spirogyra
Arkansas	−	−	−	−	+	−	−	−	+	+	−	−
California	+	+	−	−	+	+	+	−	+	+	+	+
Colorado	−	−	+	+	−	−	+	−	−	−	+	+
Connecticut	+	−	+	−	−	+	+	+	−	−	+	+
Dist. of Columbia	−	−	−	−	−	−	+	−	−	−	+	−
Georgia	−	−	−	+	−	−	−	−	−	−	−	−
Idaho	−	−	−	−	−	−	−	+	+	−	+	+
Illinois	−	−	−	−	+	+	+	+	−	+	+	+
Indiana	−	+	+	−	−	−	+	−	−	−	+	+
Kansas	−	−	−	−	−	−	+	−	−	+	+	+
Kentucky	+	+	−	−	+	+	+	+	−	−	+	+
Louisiana	−	−	−	−	−	−	+	−	−	−	−	+
Maine	−	−	+	+	−	−	−	−	−	−	−	−
Maryland	+	−	−	−	−	−	−	−	−	+	+	+
Massachusetts	−	+	−	+	+	+	+	−	+	−	+	+
Michigan	+	−	−	+	−	+	−	−	−	+	−	−
Minnesota	−	+	−	−	+	+	−	+	−	−	+	−
Missouri	+	−	−	+	+	+	+	+	+	+	+	+
Montana	+	−	−	−	−	−	−	−	−	−	+	−
Nebraska	−	−	−	−	−	+	−	−	−	+	−	+
New Hampshire	−	+	+	−	−	−	−	+	−	−	−	−
New Jersey	+	+	−	+	+	+	+	−	+	−	+	+
New Mexico	−	−	−	+	−	−	−	−	−	−	+	−
New York	+	+	+	+	+	+	+	+	+	+	+	+
Ohio	−	+	−	+	+	−	+	−	−	+	+	+
Oklahoma	−	−	−	−	−	−	+	−	−	+	−	−
Pennsylvania	+	+	+	+	−	−	+	−	+	+	+	+
Rhode Island	−	−	−	−	−	+	−	−	−	−	−	−
South Carolina	−	−	−	−	−	−	+	−	−	−	−	−
South Dakota	−	−	−	−	−	−	+	−	−	+	+	+
Tennessee	−	−	−	+	−	−	−	−	−	−	−	−
Texas	−	−	−	+	−	−	+	−	−	+	+	+
Vermont	−	−	−	−	−	−	−	−	−	+	+	+
Virginia	−	+	−	−	−	−	+	+	+	+	−	+
Washington	−	−	−	−	−	−	−	−	−	−	+	+
West Virginia	−	−	−	−	−	−	−	−	−	−	−	+
Wisconsin	−	−	−	−	+	−	+	+	+	+	+	+

[1] KARL F. KELLERMAN: "The Rational Use of Disinfectants and Algicides in Municipal Water Supplies." Paper read before the Eighth International Congress of Applied Chemistry, vol. 26, p. 241.

TABLE SHOWING QUANTITY OF COPPER SULPHATE REQUIRED TO KILL VARIOUS FORMS OF ODOR-PRODUCING ORGANISMS

Copper sulphate expressed as parts per million

Anabaena	0 09	Kirchneriella	5.00 to 10
Asterionella	0.10	Leptomitus	0.40
Beggiatoa	5.00	Microspora	0.40
Chara	0 20 to 5	Navicula	0.07
Cladophora	1 00	Oscillatoria	0.10 to 0.40
Cladothrix	0 20	Peridinium	2 00
Clathrocystis	0.10	Scenedesmus	5.00 to 10
Coelosphaerium	0.30	Spirogyra	0.05 to 0 30
Conferva	0.40 to 2	Ulothrix	0.20
Crenothrix	0 30	Uróglena	0 05
Euglena	1 00	Volvox	0 25
Fragillaria	0.25	Zygnema	0 70
Hydrodictyon	0.10		

The use of too large quantities of copper sulphate is dangerous to fish life. The following table taken from Dr. Kellerman's paper shows the safe limit for treating water with copper sulphate when certain fish are present.

TABLE SHOWING SAFE LIMIT FOR TREATING WATER WITH COPPER SULPHATE WHEN CERTAIN FISH ARE PRESENT

Copper sulphate expressed as parts per million

Black bass	2 10	Pickerel	0.4
Carp	0 30	Suckers	0.30
Catfish	0 40	Sunfish	1.20
Goldfish	0.50	Trout	0.14
Perch	0.75		

A not infrequent result following the application of copper sulphate to a water supply infested with microörganisms is the appearance of large numbers of bacteria. These are harmless forms of water bacteria that multiply rapidly, probably on account of the sudden increase in food supply caused by the decomposition of the organic matter resulting from the killing of the microörganisms.

While copper sulphate has been proposed as a bactericidal agent, its efficiency in this connection is so much less than other disinfectants, such as chlorine and its compounds, for example, that it has never come into practical use.

CHAPTER XXIX

DISINFECTION OF WATER SUPPLIES (CONTINUED)

THE CAUSE OF THE DESTRUCTION OF MICROÖGANISMS BY DISINFECTANTS

The exact cause for the destruction of microörganisms by any of the methods of disinfection described, namely by the hypochlorites, ozone or ultra-violet light, is none too well understood. Hypochlorous acid or its salts are, as has been pointed out, really agents for the production of oxygen in a very active state. Ozone, also, by its decomposition, produces oxygen which is liberated in an extremely active condition. By the passage of ultra-violet light rays through oxygen gas ozone is known to be formed. It is not difficult to conceive, therefore, that ultra-violet light rays may convert some of the dissolved oxygen always present in drinking water into ozone, which in breaking down sets free active oxygen in exactly the same manner as does the ozone prepared in ozonizers and applied as previously described.

Should this theory of the action of ultra-violet light rays prove true, then all of the methods for disinfection described can be explained as oxidation methods, and the destructive effect of even the minute quantities of oxygen produced could be attributed to the energy set free by the breaking down of the molecule of hypochlorous acid or of ozone. This free energy would seem to afford a sufficient reason for the bactericidal effects produced.

Another theory which has been advanced is that the ultra-violet rays produce hydrogen peroxide, which like ozone acts as an oxidizing agent, and which is the cause of the death of the microörganisms. Dr. von Recklinghausen contends, however, that sterilization is not produced with ultra-violet light by the formation of hydrogen peroxide in the water, as the quantity that could be formed would be too small to kill the bacteria. He also maintains that these bactericidal effects can be brought about in a water free from oxygen, and that they are not due to any chemical produced in the water, but are the result of the light itself.

Selective Action of Disinfectants and Its Effects.—It is evident that any of the disinfecting agents described may be more destructive of some microörganisms than of others. Those bacteria, for example, that may be in the spore state, will not be as susceptible to disinfecting agents as those that are not in this condition. Bacteria that do not produce spores, in which class are found most of the pathogenic forms dangerous to human beings, should be, therefore, more easily killed.

A peculiar result sometimes produced by the use of the hypochlorites is that of the reappearance in large numbers of bacteria in a water that had apparently been sterilized, or if not rendered entirely free from bacteria had had the number of organisms materially reduced. Experiments by the author, which were made under controlled conditions, reproduced this phenomenon, which is not of infrequent occurrence in water-works operation. It would appear as though either all or nearly all of the original organisms had been so adversely affected by the toxic influence of the disinfectant, that they could not grow even under favorable conditions on culture media, thus causing the conclusion to be drawn that the water was sterile or practically so; or that the less virile organisms had been actually killed, and only a few of the more hardy forms had revived and become the nuclei for a large secondary growth under more favorable environmental conditions. It is not improbable that disinfectants may exert to some extent merely a temporary inhibitory action so far as reproduction upon culture media is concerned; but it is also true that many organisms are actually destroyed, and usually those of a pathogenic character.

Secondary growths are not regarded as of any particular sanitary significance. Enormous bacterial growths of this character not infrequently follow the application of copper sulphate, where the latter is used for destroying algæ and diatoms in a water. The death and decay of this microscopic plant life merely affords the optimum conditions for bacterial activity. In other words, the equilibrium of the bacterial flora is disturbed by the action of the disinfectant, and species, which under normal conditions would only reproduce slowly, find an opportunity to multiply rapidly without hindrance from other plant and animal forms.

Tastes and Odors Produced in Water Treated with Chlorine and Its Compounds.—The taste of chlorine or its compounds in the small quantities generally employed in the disinfection of

water has been described as "medicinal," or like the taste of carbolic acid. The taste is usually detected before the odor. Only those persons who are particularly sensitive to tastes and odors will be able to detect the smaller amounts. The average observer will not taste as little as 0.4 part of chlorine per million, although 0.5 part would be more frequently noted, and 0.6 part per million would be an amount which could be generally detected.

The odor of chlorine in a bleaching-powder solution, according to Lederer and Bachmann,[1] could be identified when it contained 1.8 parts per million of "available chlorine," while it was noted in a solution of chlorine gas in water when there was present only 0.9 part per million.

While it may be true that these tastes and odors are attributable to the calcium hypochlorite or the chlorine gas added, there is some evidence to indicate that they may be due to organic compounds produced by the reaction between the chlorine and the soluble organic matter naturally present in most drinking waters. The author has made extremely disagreeable odors and tastes in an artificial leaf infusion by treating it with varying amounts of a bleaching-powder solution.

It is of interest to note that there is a very wide variation in the amounts of bleaching powder that may be used in different waters without producing disagreeable tastes and odors. Mr. F. F. Longley[2] has compiled some statistics on this point which he obtained from answers to questions sent to over 100 different water-works plants in the United States. He found that in one plant as much as 37 lb. per million gallons had not produced any taste or odor, and that in many places 20 to 30 lb. of bleaching powder per million gallons have not been noticeable. On an average about 14 lb. per million gallons did not appear to give any trouble.

For the removal of tastes and odors due to overdosing with bleaching powder, Lederer and Bachmann[3] recommend the use of sodium thiosulphate in quantities equal to one-half of the chlorinated lime applied. This chemical is cheap and adds about 40 per cent. to the cost of the bleaching powder.

[1] *Proc.* Illinois Water Supply Association, March, 1912.

[2] Report of Committee on Water Supplies of Sanitary Eng. Sect. of A. P. H. A., December, 1914.

[3] *Proc.* Illinois Water Supply Association, March, 1913.

NOTE.—Assuming bleaching powder had 33 per cent. of available chlorine, each 2.5 lb. per million gallons of water is equivalent to 0.1 part of chlorine per million; hence 25 lb. per million gallons equals 1 part per million of chlorine.

Point for Application of Disinfectants and Amounts Used.—The most effective point in a water-supply system for the application of disinfecting agents naturally varies with local conditions, with the character of the water to be treated, and with the particular method of disinfection employed. Those waters which are not subjected to any form of purification prior to disinfection will usually require somewhat greater amounts of the disinfecting agent or a longer period of contact with it than those that have been purified. For example, waters high in soluble coloring matter will use up more chlorine, more ozone, or require a longer exposure to ultra-violet light rays, than if they are colorless. The same is true of waters containing suspended matter in any considerable amount.

After a water has been disinfected it should be delivered as quickly as possible into the distribution system for consumption. Long storage periods are undesirable, but where they can not be avoided, it is advisable to store in covered reservoirs, unless the expense involved is prohibitive. There is, of course, more danger from atmospheric and surface drainage contamination in open than in covered reservoirs, and once a water has been prepared for consumption, it should not be subjected to conditions that may again render it unsafe. On the other hand, sufficient time must elapse for the full effect of the disinfectant to be produced. Fortunately this period is quite short for practically all of the disinfecting agents described. Practical disinfection with chlorine and its compounds may be effected in 15 to 30 min., and more than 1 hr. would probably never be required. Provision for about 1 hr. of contact between the ozonized air and the water to be treated was provided for in one of the German ozone plants, and this is doubtless sufficient in most cases. Where ultra-violet light is used only a few minutes exposure in the light zone is claimed to be necessary to produce absolute sterility.

Mr. Francis F. Longley in the report previously referred to states that 40 per cent. of the water supplies about which they obtained information concerning the use of the hypochlorites for disinfection had no storage following disinfection; 18 per cent. of them had less than 1 hr.; 9 per cent. of them had from 1 to 3

hr.; 5 per cent. from 3 to 12 hr.; 11 per cent. from 12 to 24 hr.; and 17 per cent. more than 24 hr.

When a water supply is filtered, the most economical point to apply the disinfecting agent is in the filtered-water reservoir. In many rapid sand filter plants, no common point for the mixed effluents of all the filters is reached until the water leaves the filtered-water reservoir. In such cases as this there is no alternative but to apply the disinfectant at this point, and depend upon the contact period in the pipe line to produce the effect desired. In the case ultra-violet light rays are used, there must also be some common point past which all the filtered water flows, and where exposure to the light is possible.

Mr. Longley found that 37 per cent. of the cities giving replies to his questions used a disinfectant without other treatment, and that the balance used it as an adjunct to some other form of purification, usually in connection with filtration. In 57 per cent. of the plants it was used as a final treatment in connection with filtration; in 26 per cent. it was used after coagulation or sedimentation and before filtration, and in 17 per cent. it was applied before coagulation and filtration. In about 90 per cent. of the plants the use of the disinfectant was continuous, and in the remainder intermittent.

The use of chemical disinfectants prior to coagulation and filtration requires the application of larger amounts than would be the case if they were applied to the clear filtered water. There would appear to be little justification for this procedure, unless odors and tastes were lessened, or prevented in the filtered water. The danger of destroying the colloidal coating upon the sand grains of the filters by using too large amounts of hypochlorites in the applied water is a possibility that the author knows from experience can happen.

Bacterial Reduction by Disinfection.—The efficiency of disinfection processes depends, as in other methods of purification, on the care with which the process is carried on, and with the character of the water being treated. In general, a reduction in the number of bacteria to less than 10 per cubic centimeter can be usually expected. In some plants no effort is made to reduce the bacteria to the lowest possible number, since this procedure might require too great an amount of the disinfecting agent, which would unduly increase the cost without rendering the water much safer from a sanitary standpoint. Where chlorine

or hypochlorites are employed, the difficulty from tastes and odors from excessive amounts of these agents is always a matter to be given consideration. The reduction in the number of B. coli, as indicated by the usual tests for this organism, should be given due weight in the interpretation of the effects produced by disinfection.

Relative Effects of Disinfectants on Various Species of Bacteria.—A large amount of experimental work has been carried on during the past few years to determine the ability of various bacteria to withstand the action of disinfectants. The resistance of spore-bearing bacteria has been especially studied. Mr. N. S. Hill, Jr., states that bleaching powder applied as a disinfectant to water does not destroy the spore-bearing bacteria, such as B. aerogenes capsulatus, B. butyricus, and B. cadaveris sporogenes. Lederer and Bachmann found that 25 parts per million of available chlorine effected only a 3 per cent. reduction in the number of B. subtilis in 15 min., and that 400 parts per million of chlorine acting for 15 min. would produce a 95 per cent. reduction. B. mesentericus rosei and B. mesentericus fuscus required a contact of 15 min. with 5 parts per million of chlorine to be reduced in number 80 per cent. and 26.4 per cent., respectively. These

PERCENTAGE REDUCTION[1] ON DISINFECTION OF THE GROUP OF INTESTINAL BACTERIA WITH CALCIUM HYPOCHLORITE BY A CONTACT OF 15 MIN.

Bacteria per c.c.	160,000	9,500	3,000	8,000	180,000	180,000	500
Parts per million of chlorine applied	B. cloacæ	B. fecal. alkalig.	B. paratyph.	B. prot. mirabilis	B. enteritidis	B. lactis aerog.	B. choleræ suis
0.1	......	99.98	......	27.3	......	......	......
0.2	99.69	99.99	99.97	45.5	99 83	99.17	95.8
0.3	99.75	100.00	100.00	63.7	99.98	99.98	100.0
0.5	100.00	......	......	72.7	100.00	100.00	
0.7	......	....	......	63.7			
1 0	......	.....	......	63.7			
3.0	......	......	... ..	90.9			
5.0	......	......	.... .	90.9			

[1] *Proc.* Illinois Water Supply Association, March, 1912.

same investigators found that intestinal bacteria were much more susceptible to disinfectants as the preceding table will show.

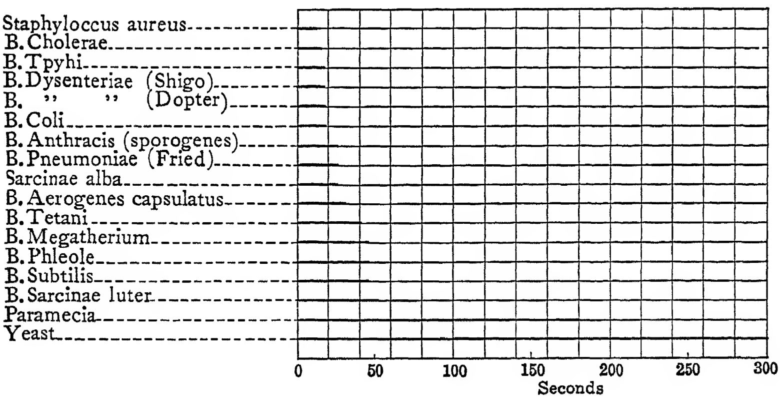
Similar experimental data are available for the effects of ozone. The following table taken from the Report of the Imperial Board of Health of Germany, Book XVIII, vol. 3, 1902, gives the results of tests made by Drs. Ohlmüller and Prall.

BACTERIA PER CUBIC CENTIMETER

Before ozonization	After ozonization		Species
	Ordinary forms	Pathogenic forms	
38,330	8	0	B. choleræ
16,590	9	0	B. typhi

N. B.—Ordinary Spree and aqueduct water infected with cholera germs, and boiled water infected with typhoid fever germs.

Dr. Max von Recklinghausen[1] has recently given some data showing the length of time required to kill various species of bacteria by exposure to rays of ultra-violet light.



It was found in other experiments undertaken by Dr. von Recklinghausen, that water infected with B. coli could be rendered sterile by the light rays from a 220-volt 3-amp. mercury vapor quartz lamp acting for 1 sec. at a distance of 4 in.; in 4 sec. at a distance of 8 in.; in 15 sec. at a distance of 16 in.; and in 30 sec. at a distance of 24 in.

[1] *Jour. New England Water-works Assn.*, June, 1915.

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The variability in resistivity to ultra-violet light appears to be considerably less than in the case of chemicals like chlorine and ozone.

Practical Results Obtained by Disinfection.—The actual results obtained with disinfecting agents in practice are shown by the following data taken from different sources.

During the first 4 months of 1913, there were used at the Cincinnati filtration plant over 10 tons of bleaching powder. This amount was applied to over 500,000,000 gal. of water, and was equivalent to practically 0.14 part per million of available chlorine. The disinfectant was applied in the form of a solution to the filtered water. The results produced upon the bacterial content of the water are shown in the following table.

Average Number of Bacteria per Cubic Centimeter

1913	Ohio River water	Settled water	Filtered water	Disinfected water
January	61,500	16,500	470	70
February	16,140	3,550	150	41
March	25,120	4,060	110	14
April	9,575	3,260	80	26

The average removal of bacteria by settlement and filtration was 99.3 per cent., and by disinfection 81 per cent.

At the Cincinnati plant in 1915 liquid chlorine was used during the whole year. The percentage removal of the bacteria as a whole and of the B. coli for each step in the process of purification is given in the following table.

Percentage Removal of All Bacteria and of B. Coli for Each Step in Process

<table>
<tr><th rowspan="2">Process</th><th colspan="4">All bacteria</th><th colspan="2">B. coli</th></tr>
<tr><th colspan="2">Based on number in raw water</th><th colspan="2">Based on number applied at each step in process</th><th>Based on number in raw water</th><th>Based on number applied at each step in process</th></tr>
<tr><td>By plain sedimentation</td><td colspan="2">81.17</td><td colspan="2">81.17</td><td>85.81</td><td>85.81</td></tr>
<tr><td>By coagulation</td><td>15.34</td><td rowspan="2">18.12</td><td>81.05</td><td rowspan="2">96.28</td><td rowspan="2">13.67</td><td rowspan="2">96.32</td></tr>
<tr><td>By filtration</td><td>2.78</td><td>79.86</td></tr>
<tr><td>By disinfection</td><td colspan="2">0.50</td><td colspan="2">71.03</td><td>0.35</td><td>66.80</td></tr>
</table>

The quantity of chlorine applied at the above plant averaged 0.12 part per million.

As an illustration of the effect of bleaching power on an effluent from slow sand filters, the result of treating the water at the Torresdale filter plant in Philadelphia is of interest. Fron 0.16 to 0.33 part per million of available chlorine was used during the 2 years tabulated.

	Filtered Water Basin		Tap at Lardner's Point	
	Per Cent. Positive Tests for B Coli			
1911	1 c.c.	10 c.c.	1 c.c.	10 c.c.
Untreated water, May–Nov., inclusive .	14.5	58.0	10.0	48.6
Treated water, Jan., Apr. and Dec., 1912..	4.0	13.3	2.4	12.9
Treated entire 12 months, 1913...........	4.1	22.0	2.0	16.6
Treated for 7 months, Jan.–July, inclusive..	0.9	3.3	0.6	6.2

Over 0.16 part of chlorine used continuously.

An example of the effect of bleaching powder on the water of Lake Erie, used without purification other than disinfection, as the water supply of Cleveland, Ohio, is given by Mr. D. D. Jackson in his Report on the Sanitary Condition of the Cleveland Water Supply (1912). The amount of available chlorine applied varied from 0.50 to 0.86 part per million.

1912	Bacteria per cubic centimeter	
	Untreated water	Treated water
March..	18	7
April...	31	19

1912	B. coli per cent. of times found in dilution indicated					
	Untreated water			Treated water		
Cubic centimeter tested.... .	0.1	1.0	10.0	0.1	1.0	10.0
February....................	0	6	53	0	0	0
March.....................	0	13	52	0	0	16
April.......................	0	17	43	0	0	7

The effect of ozone on bacteria, and incidentally on the reduction of color in a water, is well illustrated by tables taken from a paper by Mr. S. T. Powell[1] describing the treatment of an unfiltered colored water obtained from the Herring Run supply of the Baltimore County water system.

According to Mr. Powell the following table illustrates the bacterial efficiency of ozone at low concentrations when a good contact between the gas and the water is effected by proper mixing. The water treated was unfiltered.

Bacteria per c.c.		Percentage removed	Temperature air, deg. F.	Humidity	Ozone concentration, grams per cubic meter of air
Raw water	Ozonized water				
2,720	25	99.1	30	90	1.05
1,600	16	99.0	24	80	1.77
1,400	8	99.5	34	76	1 26
1,580	56	96.4	54	70	1.27
740	16	97.8	78	64	0 58
400[2]	40	90.0	80	76	0.84

The effect of the ozone on vegetable coloring matter in the water with low concentrations of the gas and under varying weather conditions is shown by the following table.

Color in parts per million		Percentage removed	Temperature air, deg. F.	Humidity	Ozone concentration, grams per cubic meter of air
Raw water	Ozonized water				
35	15	57.2	48	88	1.69
30	20	33.3	40	58	2.54
28	18	35.7	68	68	0.76
28	20	28.5	80	68	0.90
28	20	28.5	80	76	0.84
28	20	28.5	78	80	0.63
50	40	20.0	82	80	0.63

The following bacterial results were obtained some years ago in testing ultra-violet ray apparatus designed to be used to ster-

[1] "The Use of Ozone as a Sterilizing Agent for Water Purification." S. T. Powell: *Jour. New England Water-works Assn.*, March, 1915.

[2] High turbidity in raw water.

ilize the water supply of Marseilles, France. The water came from the River Durance and was filtered before sterilization. The volume of water treated was about 158,500 gal. in each test.[1]

Water treated in 24 hr, gallons	Watt-hours per 1,000 gal.	B coli per 100 cubic centimeters	Bacteria per cubic centimeter	B coli[2] per 100 cubic centimeters	Bacteria[3] per cubic centimeter
		Before sterilizing		After sterilizing	
158,000	98.4	....	160	...	0.80
158,000	98.4	100–200	240	0	2.00
158,000	98.4	....	20	0	1.00
158,000	98.4	500	37	0	0.00
147,940	106.0	80	20	0	0 07
158,400	98.4	50	48	0	0.00
158,400	98.4	50	23	0	2.00
158,400	98.4	200	29	...	4 20
158,400	98.4	500–1,000	51	0	0.17

Dr. Max von Recklinghausen gives the following table of bacterial results, obtained by sterilizing with ultra-violet light, in the *Journal of the New England Water-works Association* (June, 1915).

	Before sterilization, bacteria per c.c.	After sterilization, bacteria per c c.	Operator
	8,000	0.0	Westinghouse Laboratory.
	2,840	0 6	
Sewage bacteria...	80,000	0.0	Glaser, Austrian Army Medical Service.
Sewage bacteria...	11,000	12.0	Burgess, London.
Sewage bacteria...	3,740	0.0	Thresh and Beale, London.
B. coli communis.	35,000	0.0	Jurist, New York.
Sewage bacteria...	273,000	0.0	Westinghouse Laboratory.
B. typhosus.......	25,000	0.0	Philadelphia Clinical Laboratory.
Sewage bacteria..	20,000	0.0	Bengal Sanitary Committee.

These tests were made with somewhat different types of apparatus. The first five tests were made with the B-2 type of sterilizer, the sixth and seventh tests with the B-5 type, and the eighth test with the C-3 type.

[1] *Eng. Record*, vol. 62, Dec. 10, 1910.

[2] Mean of results from two to five samples.

[3] Mean of results from two to seven samples.

Hygienic Effects of Sterilization.—The marked effect of disinfection upon the sanitary quality of drinking waters, as measured by the stopping of epidemics of water-borne diseases like typhoid fever, or by the permanent reduction of endemic typhoid, has been most striking. The following table compiled by Mr. C. A. Jennings[1] brings out this effect quite clearly.

City	Average typhoid fever death rates per 100,000 persons				
	Began using calcium hypochlorite	Before using		After using	
		Period	Rate	Period	Rate
Baltimore . . .	June, 1911	1900–10	35.2	1912–13	22 8
Cleveland . . .	Sept., 1911	1900–10	35.5	1912–13	10.0
Des Moines. . .	Dec., 1910	1905–10	22.7	1911–13	13 4
Erie.	March, 1911	1900–10	38.7	1912–13	13 5
Evanston.	Dec., 1911	1907–10	26.0	1912–13	14.5
Jersey City. . .	Sept., 1908	1900–07	18.7	1909–13	9 3
Kansas City	Jan., 1911	1900–10	42.5	1911–13	20.0
Omaha.	May, 1910	1900–09	22 5	1911–13	11.8

Of the eight cities tabulated, the maximum reduction in the typhoid death rate, 72 per cent., was in Cleveland, and the minimum, 35 per cent., in Baltimore. The average reduction was 51 per cent. The above cities contain a combined population of over 2,000,000 people.

Cost of Apparatus for Disinfection.—The cost of the equipment for disinfection varies widely, and depends on the particular process employed, and whether a simple or elaborate design is adopted. Local conditions also naturally affect the cost, as for example, the separate housing of the equipment in some instances and not in others, the carrying of liquid disinfectants longer or shorter distances, or the provision for independent power plants, as in the case of ozone and ultra-violet ray plants, instead of adapting or using existing sources of power.

The cost of the equipment for applying bleaching-powder solutions includes usually the cost of the solution tanks, orifice tanks or apparatus, water-supply pipe lines, solution pipe lines and sewer lines for the disposal of sludge. Pumps may be required in some cases to handle the hypochlorite solution. A building for the apparatus may or may not be necessary. Frequently

[1] *Proc.* Illinois Water Supply Association, March, 1914.

space in or slight modifications to buildings already on the ground obviate the necessity for separate structures.

The cost of bleaching-powder disinfecting plants, based upon a capacity for treating 1,000,000 gal. of water per day may vary from \$25 to \$500. The larger the capacity of the plant the smaller the unit cost, and *vice versa.* For plants in which 5,000,000 gal. of water or less are to be treated each day, a total outlay of \$300 to \$500 should be ample if no building to contain the apparatus is included. For plants having a capacity of 5,000,000 to 20,000,000 gal. per day, the cost will not probably exceed \$100 per million gallons of daily capacity, and for plants treating above 20,000,000 gal. daily, the cost may drop as low as \$50 or \$60 per million gallons of daily capacity.

Apparatus now on the market for applying liquid chlorine as a gas does not vary materially in size or design for any given type, whether it is intended to be used to treat a few hundred thousand gallons, or many millions of gallons of water daily. Single units cost approximately from \$350 to \$700 each. A recent installation for a large city which was intended to treat 60,000,000 gal. of water per day and which consisted of three units, cost \$2,000. Ten chlorinators placed in the purification plants in Philadelphia several years ago cost \$9,750.

The cost of ozonizing plants properly includes the ozonizers with the power plant to operate them, the refrigerating plant to cool and dry the air, and the sterilization towers. The cost for such a plant will naturally vary greatly, depending upon the local conditions. Mr. S. T. Powell in estimating the operation costs for the Herring Run plant of the Baltimore County Water & Electric Co. takes as a basis \$5,000 per million gallons as the original cost for 10,000,000-gal. units or for larger installations, and which would include roughing filters and a refrigerating plant.

The cost of ultra-violet ray apparatus is about \$750 per thousand gallons per hour capacity for the larger machines, and rising to \$900 for the smallest. Large installations will, of course, cost much less in proportion to capacity. The estimated cost for the Niagara Falls, N. Y., plant, which would have treated 16,000,000 gal. daily, was \$19,800 for the canals and lamps, and \$2,200 for transformers and building to house the same. This plant has not been installed at Niagara Falls, although the project was given careful consideration and study.

Cost of Operation of Disinfection Plants.—The cost of operating disinfection plants varies widely, as would be expected from the differences in the initial costs of the plants, and of the various agencies which they employ. The character of the water being treated also affects the cost, since the amounts of chemicals used or of power expended to produce disinfection is obviously not equal for all waters.

For disinfecting with bleaching-powder solutions the cost per million gallons of water treated will vary in the United States from 10 to 50 cts., the average cost being about 25 cts. These variations are due in part to the range in the cost of the bleaching powder in different parts of the United States. Between the Atlantic coast and the Mississippi River the normal price for bleaching powder is about $1.78 per hundred pounds, while from the Mississippi River to the Pacific coast, the cost ranges from $2.50 to $3 per hundred weight. The smaller plants buy in relatively small packages, and hence must pay a higher price than do the large plants buying in large drums and in considerable quantities. The depreciation in the value of bleaching-powder plants is high, and, of course, adds to the operation and maintenance costs.

Where liquid chlorine is used, the costs do not differ much from those incurred by the use of bleaching powder. The depreciation in value of apparatus is not, perhaps, quite as great as where bleaching powder is employed, but, nevertheless, must be taken into account in any true estimate of the cost of using this agent. With a normal price of about 8 cts. per pound for liquid chlorine, the cost per million gallons for disinfection at the Cincinnati filtration plant varies from 8 to 12 cts. for the chemicals alone.

The cost of disinfection with ozone is much higher than where the hypochlorites or liquid chlorine are used. To produce a concentration of 2.5 grams of ozone per cubic meter of air at the Petrograd plant in Russia, the ozonizers consumed 57-kw.-hr. per million gallons of water treated, and the compression of the ozonized air in order to force it into the towers required 76 kw.-hr. more of electrical energy. In addition to these costs are those resulting from drying and cooling the air by refrigeration. It was estimated that the total cost of filtration and disinfection with ozone at the Petrograd plant was $15 to $17 per million gallons, of which about one-half was chargeable to the ozone process.

Mr. S. T. Powell of the Baltimore County Water & Electric Co., estimates that the cost of ozone sterilization in this country will be between $2.50 and $4 per million gallons. His estimate

> "is based upon a maximum installation cost of $5,000 per million gallons on 10,000,000-gal. units, or larger installations, which includes roughing filters and refrigeration plant, interest on the investment at 5 to 5½ per cent., depreciation at 5 per cent., current at a maximum of 3 cts. per kilowatt-hour, as well as supervision and management."

The consumption of electrical energy in operating ultra-violet ray apparatus is estimated by Dr. Max von Recklinghausen not to exceed 100 kw.-hr. per million gallons. The maintenance cost of the lamps must be added to the cost of the current. Lamp renewals may cost from 50 to 60 cts. per million gallons. Even with a low cost for the electrical current, it would not appear as though disinfection by this process could cost less than 75 cts. per million gallons. With current at 1 ct. per kilowatt-hour and a lamp renewal cost of 50 cts., the cost per million gallons would be $1.50.

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CHAPTER XXX

THE REMOVAL OF DISSOLVED MINERAL MATTER FROM WATER

WATER SOFTENING

The origin and general character of the mineral matter dissolved in natural waters were discussed in Chapter II.

The amount of mineral matter dissolved in natural waters varies greatly. The reasons for desiring to reduce the quantity of these mineral compounds in solution are to render the water more suitable for domestic use, or to make it better fitted for industrial purposes. If the water is being used as a public water supply, both of these reasons may be quite properly urged as sufficient grounds for installing some form of purification.

It is obvious that the nature of the dissolved impurities determines the character of the purification method to be employed. It is also apparent that means may be used for treating a water for industrial purposes, that are not permissible in connection with a public water supply. The hardness of a water, that is caused by salts of calcium and magnesium, does not in itself make it unfit for drinking purposes. In fact, a small amount of hardness in a drinking water is probably beneficial to those who consume it, and does not render it unfit for most industrial uses. Only when the amount of the dissolved salts of calcium and magnesium become excessive is it necessary to resort to methods of purification. On the other hand, even comparatively small amounts of iron, manganese, or of free acid are objectionable for domestic purposes, and for most industrial uses. Waters containing large amounts of saline matter are unfitted for both drinking and technical purposes. For example, great quantities of the soluble salts of sodium, more especially the chloride, sulphate and carbonate of this element render a water practically worthless, so far as being able to make it fit for most purposes by any methods of purification now known, other than by distillation. This latter means of removing the dissolved salts is obviously of limited value where considerable quantities of water are to be purified.

According to Deussen and Dole[1] waters that have a hardness that does not exceed 200 parts per million, or that do not contain enough mineral matter to have a disagreeable taste, are acceptable for drinking and cooking. A hardness greater than 1,500 parts per million renders a water undesirable for cooking, while a water containing approximately 250 parts per million of chloride has a slightly salty taste. Somewhat less of the carbonate and more of the sulphate may be tolerated. However, a drinking water containing more than 300 parts per million of carbonate, 1,500 parts of chloride, or 2,000 parts of sulphate is unhealthful to most persons. On these bases a sodium chloride water showing more than about 3,000 parts per million of mineral matter, or a sulphate water showing more than 3,500 parts of mineral matter is reasonably classed as unfit for domestic use. The drinking of such water is usually characterized by intestinal disturbances.

The expense to a community using a hard water for soap alone is considerable, and may be sufficient in itself to warrant the cost of softening. Lessened troubles with plumbing for hot-water systems, the reduction of scale in boilers, and the greater value of a soft water for many manufacturing processes, are all excellent reasons for resorting to methods of purification for reducing excessive quantities of lime and magnesia, or eliminating compounds of iron and manganese.

Removal of Salts of Calcium and Magnesium.—The process of removing the bicarbonates, sulphates and chlorides of calcium and magnesium is commonly termed "water softening," that is the reduction of the "hardness" of a water produced by the presence of these salts in solution. Temporary hardness,[2] caused by the bicarbonates of calcium and magnesium, and in some cases iron, is that portion of the "total hardness" of a water that may be removed by boiling. The remainder of the total hardness is commonly known as the "permanent hardness" of the water, and is almost always entirely due to the sulphates and chlorides of calcium and magnesium. In some cases nitrates of these elements may contribute more or less to the permanent hardness

[1] ALEXANDER DEUSSEN and R. B. DOLE: "Ground Waters in LaSalle and McMullen Counties, Texas." Water Supply Paper No. 375, U. S. Geological Survey.

[2] For a scientific classification of waters with reference to their dissolved mineral constituents, the reader is referred to Chapter II. The terms used above are those commonly employed in describing this class of waters, and are not strictly accurate.

of the water. Liquid waste products from industrial processes that contain free mineral acids or sulphates of iron, aluminum or manganese are not infrequently discharged into streams, and may create a "hardness" in the water that renders it unfit for almost any use, unless the "acidity" produced is neutralized. It is evident, therefore, that water-softening processes must be adapted to the particular character of the compounds producing "hardness," and that different reagents are required to obtain the results desired.

Chemistry of Water Softening.—The solubility of the carbonates of calcium and magnesium in water is greatly increased by the presence of carbonic acid in solution. Under these conditions the carbonates are in reality bicarbonates of these elements. This free and loosely bound carbonic acid is easily removed by boiling, but may also be removed by caustic lime. The use of lime for removing the free and half-bound carbonic acid, thereby converting the bicarbonates into normal or monocarbonates, is the basis of one of the oldest and most efficient water-softening processes known. The chemical reactions involved are as follows:

1. $H_2CO_3 + Ca(OH)_2 \rightleftarrows CaCO_3 + 2H_2O$.
2. $CaCO_3, H_2CO_3 + Ca(OH)_2 \rightleftarrows 2CaCO_3 + 2H_2O$.
3. $MgCO_3, H_2CO_3 + 2Ca(OH)_2 \rightleftarrows 2CaCO_3 + Mg(OH)_2 + 2H_2O$.

There are two features of these reactions which should be noted, namely, that the caustic lime added is itself precipitated as the carbonate of calcium, the same as is the calcium carbonate deprived of its half-bound carbonic acid; and that the magnesium is precipitated as the hydrate. The molecular proportions as shown by the above reactions must exist if the greatest reduction possible is desired. On account of the slight solubility of calcium carbonate in water, when no free or half-bound carbonic acid is present, the complete precipitation of the lime is impossible. Magnesium carbonate in the absence of free and half-bound carbonic acid is even more soluble in water than calcium carbonate, and unless the caustic lime applied is in sufficient amount to produce magnesium hydrate, a considerable quantity of the carbonate still remains dissolved. The solubility of the normal carbonates of lime and magnesia, and the slow precipitation of these salts as the reaction approaches theoretical completion, accounts for the residual alkalinity which waters softened with

caustic lime always exhibit. The so-called "after-precipitation effects" in waters softened with lime and soda ash, are due to the physical properties of the precipitates produced by the softening reactions. It is simply a case of the slow deposition of salts that have been in colloidal solution.

The removal of permanent hardness may be accomplished by another reagent, namely, sodium carbonate or soda ash. The reactions which take place are as follows:

1. $CaSO_4 + Na_2CO_3 \rightleftarrows CaCO_3 + Na_2SO_4$.
2. (*a*) $MgCl_2 + Na_2CO_3 \rightleftarrows MgCO_3 + 2NaCl$.
   (*b*) $MgCO_3 + Ca(OH)_2 \rightleftarrows Mg(OH)_2 + CaCO_3$.

The reactions involving sulphates, chlorides and nitrates of calcium and magnesium, and sodium carbonate, all consist of an exchange of acids, whereby the insoluble carbonates of these bases are precipitated, and a corresponding amount of the soluble sulphates, chlorides and nitrates of sodium remain in solution. The soluble salts are not reduced by the treatment, but the less objectionable salts of sodium are substituted for those of calcium and magnesium.

Free acid or the sulphates of iron, aluminum and manganese may be neutralized with sodium carbonate. The hydroxides of the bases just mentioned are formed, and on account of their insolubility may be easily removed by filtration. In the case of iron and manganese, however, oxidation of the hydroxides first formed are usually necessary in order to effect complete precipitation.

Reactions between Natural and Artificial Zeolites and Hard Waters.—By the action of hard waters on certain complex silicates of aluminum and sodium, an exchange of the calcium or magnesium for the sodium of the zeolite may be effected. The soil studies of Dr. R. Gans led him to divide zeolites containing aluminum into two general classes, which he describes as: (1) "double silicates of aluminum," and (2) "aluminate-silicates." The first class of zeolites exhibits little absorptive power for any exchange of bases as noted above. The second class of zeolites, however, appears to possess this property in quite a marked degree. Gans was able to produce artificial zeolites which showed even greater absorptive power than the natural zeolites. The artificial zeolite of Gans has the following probable composition:

$$2SiO_2.Al_2O_3.Na_2O + 6H_2O.$$

Under the trade name of "Permutit" this artificial zeolite has

been placed upon the market as a water-softening agent. The nature of the reaction between this compound and a water containing compounds of calcium and magnesium may be expressed as follows:

1. $2SiO_2.Al_2O_3.Na_2O + CaCO_3, H_2CO_3 \rightleftarrows 2SiO_2.Al_2O_3.CaO + 2NaHCO_3$.

2. $2SiO_2.Al_2O_3.Na_2O + MgSO_4 \rightleftarrows 2SiO_2.Al_2O_3.MgO + Na_2SO_4$.

The zeolite may be regarded as a solid reagent, exchanging one of its constituents for a base in the solution on which it is acting. Since the zeolite remains insoluble, it may be regenerated by a reversal of the reaction. This is accomplished by the action of sodium chloride, a strong solution of which is permitted to react on the solid zeolite for a period equal to that to which it was subjected by the hard water. The sodium of the salt substitutes itself for the previously absorbed calcium or magnesium, and the resulting chloride of the alkaline earth base goes into solution, and may be washed out with clean water. The reversible character of the reactions is simply the result of molecular concentration, or as it is sometimes called the law of mass action.

The reaction between the sodium chloride and the zeolite in which the sodium has been replaced by the alkaline earth base calcium or magnesium may be shown as follows:

$$2SiO_2.Al_2O_3.CaO + 2NaCl \rightleftarrows 2SiO_2.Al_2O_3.Na_2O + CaCl_2.$$

Artificial zeolite as prepared by Gans was made by fusing together feldspar, kaolin, silica and sodium carbonate in definite proportions. After extracting the melt with water, the porous material which remained was found to contain 46 per cent. of silica, 22 per cent. of alumina, 13.6 per cent. of sodium oxide and 18.4 per cent. of water.

Other Chemical Reagents Used in Water Softening.—The reagents described above are safe and suitable for softening a water for domestic purposes. They are also commonly employed for reducing the hardness of waters that are to be used for technical purposes. There are, however, quite a number of effective chemical agents that may be used for softening waters for technical purposes that could not be safely employed in treating waters that were to be used for drinking. For example, caustic soda, the hydrate, carbonate or aluminate of barium, the silicate or oxalate of sodium and tri-sodium phosphate are all chemicals that have been used as water-softening agents. Aqueous vege-

table extracts of oak, chestnut, logwood and quebracho are also employed on account of the tannin which they contain.

For assisting the softening of water in conjunction with heat, principally for the purpose of modifying the physical properties of the precipitates, numerous mechanical agents are used, such as graphite, talc, petroleum oils, and gelatinous or starchy substances like Irish moss and dextrine. These materials are usually employed in connection with some chemical agent such as soda ash or caustic soda. Many of the above-mentioned substances form the bases of the numerous "boiler compounds" so commonly employed in softening water within a steam boiler. Although the function of a boiler is to convert water into steam, it unfortunately too often has to play the rôle of a "softening tank" as well, when only hard waters are available for steam raising purposes. Unless a water contains only a comparatively small amount of the salts of calcium and magnesium, their precipitation by any process should be carried out in suitable apparatus outside of the boiler instead of within it. The boiler may then be kept free from scale or deposits, however they may be formed, and can be operated at its highest efficiency.

The object in using practically any of the reagents named is to produce a deposit of such a character that it will not form a scale on the iron within the boiler. The carbonates of calcium and magnesium, and the hydrate of the latter element are usually precipitated within the boiler in a finely divided condition, and do not adhere to the iron very closely. On the contrary, the sulphate of calcium produces an extremely adherent and porcelain-like scale, which forms a non-conducting heat layer on the iron, and thereby prevents the transfer of heat from the iron to the water. Overheating of the iron not infrequently results, therefore, thus ruining the tubes or sheets of the boiler, and causing leaks and possible blowouts of a much more dangerous character. If preliminary treatment of the water is effected in special apparatus, such as softening tanks and boiler feed-water heaters, these troubles with the boiler may be practically eliminated.

The corrosive character of waters containing considerable amounts of magnesium chloride, or of magnesium salts in conjunction with sodium chloride is due to the formation of hydrochloric acid at the high temperatures that may be produced within a steam boiler. Some of the magnesium chloride is hydrolyzed,

decomposing into magnesium oxide and hydrochloric acid. The latter passes into the steam, while the oxide remains in the water. For this reason sea water can not be used in ships' boilers.

The excessive amounts of dissolved salts found in some natural waters, as for example in certain mineral water springs, or in the "alkali waters" of the Western plains, can not usually be removed by any method of purification now known, other than by distillation as previously noted. The use of such waters in boilers may produce frothing and foaming, and thus become the cause for the evils arising from the passage of water mixed with steam into the steam pipes. Treatment of such waters with chemical agents can not usually effect any reduction in the total amount of dissolved solids, although it may be able to change their character to some extent. For example, the use of barium hydrate in a water containing large amounts of sulphates will precipitate the latter as barium sulphate, but leaves the equivalent amounts of the hydrates of the bases deprived of their sulphate ion.

Mr. R. B. Dole[1] states that:

"In locomotive service, it is in general economical to treat water containing 250 to 850 parts per million of incrustants, and to treat those containing less than 250 parts if the scale formed contains much sulphate. As the incrusting solids may commonly be reduced to 80 or 90 parts per million, the economy of treating boiler waters deserves consideration in a region where many supplies contain 300 to 500 parts per million of incrusting matter.

"The amount of mineral matter that makes a water unfit for boiler use depends on the combined effect in boilers of the softening reagents used with such waters and of the constituents not removed by softening. Sodium salts added to remove incrustants or to prevent corrosion increase the foaming tendency, and this increase may be great enough to render a water useless for steaming. It is not of much benefit to soften a water containing more than 850 parts per million of non-incrusting material and much incrusting sulphate. Trouble from priming in locomotive boilers begins at a concentration of about 1,700 parts per million of foaming constituents, and the limit of safety for stationary boilers is reached at a concentration of about 7,000 parts. Though waters containing as high as 1,700 parts per million of foaming

[1] U. S. Geological Survey, Water Supply Paper No. 398.

Proc. Am. Ry. Eng. and Maintenance of Way Assoc., vol. 8, p. 601, 1907.

Proc. Am. Ry. Eng. and Maintenance of Way Assoc., vol. 6, p. 610, 1905.

constituents have been used, it is usually more economical to incur considerable expense in replacing such supplies by better ones."

Practical Methods Employed in Water Softening.—The softening of large volumes of water with a chemical reagent necessitates the proper and thorough mixing of the reagent with the water, the requisite period of time for the reaction to complete itself, and provision for the adequate disposal of the waste products of the reaction. The control of the temperature conditions under which the reaction should proceed is highly desirable, but is not usually practicable.

Application of Chemical Reagents.—The general methods and apparatus employed in applying chemicals have been discussed in Chapter XXII. Only those features relating more especially to water softening with lime and soda ash, and to the technical features of water softeners of this type need be described at this point.

The lime is applied to the raw water as milk of lime or as a lime-water solution. The latter method is undoubtedly the one to be preferred, but is not so generally used on account of the large storage space required for the lime-water solution. Milk of lime suspensions must be constantly stirred if a fairly uniform strength of the mixture is to be maintained. Where soda ash is also used, it is not infrequently dissolved with the lime in the same tank. The mixture, therefore, actually consists of a suspension of hydrated lime in a caustic soda solution. As a by-product of the reaction calcium carbonate will, of course, be also found in suspension.

The methods of measuring the quantity of applied chemical solution are varied. Submerged orifices, proportional feed pumps, Sutro weirs, and even dry feeding of the chemicals are all in actual use. This class of devices was described in a preceding chapter. Another automatic arrangement not infrequently used in water-softening apparatus, more especially in those machines which soften water for industrial purposes, consists of a double-compartment oscillating tank or bucket, which measures out the hard water, and which actuates valves admitting a definite dosage of the chemical reagent solution at each tipping of the tank. The chemical solution is kept in a semi-cylindrical vessel directly above the tipping tank. The hard water flows first into one compartment of the tipping bucket, which, when full, tips

and discharges the water it contained. This movement brings the other compartment under the supply pipe and the operation is repeated. The movement of the bucket in tipping raises by means of a cam a stopper valve in the bottom of the chemical supply tank. This valve is so adjusted that it allows the required amount of chemical solution to pass before it closes again, being assisted in so doing by a weight and spring. The milk of lime soda ash solutions are kept mixed by a stirrer actuated by the motion of the tipping bucket. The movement of the bucket has also been used to operate chemical feed pumps.

Mixing the Chemicals with the Hard Water.—Since the intimate mixing of the chemical reagents with the water is necessary to produce the desired exchange of acids and bases, this phase of the process is of the greatest importance. This is accomplished in two ways, namely, by passing the treated water through highly baffled compartments, or by mechanical agitation by means of stirrers. In a few plants agitation is effected by using compressed air. The baffling method is the only one available where large volumes of water are handled, and is exemplified by the plants at Columbus, Ohio, Grand Rapids, Mich., and New Orleans, La. Mechanical agitation is employed at Owensboro, Ky., where there are four mixing chambers containing four sets of agitators.

A mixing period of about 1 hr. is used at several of the large plants, while in small water-softening machines of the continuous type only a few minutes is allowed. The best period for agitation doubtless varies with different waters, and considerations of cost for storage in tanks or reservoirs will usually set the limit beyond which it is not practical to go. The colloidal character of the precipitates that are formed requires more or less agitation for their complete deposition. A short period of violent agitation is probably better than a longer period of less active agitation. Mixing of previously formed sludge deposits with freshly treated water, by pumping the latter into the bottom of a settling tank through a perforated pipe lying in the sludge, has also been used to facilitate the deposit of the colloidal precipitates produced by the reactions.

The Settling of the Precipitates Formed.—The finely divided character of the precipitates, due largely to their colloidal character, makes their deposition a rather slow process. In large plants, where a continuous flow of water must be maintained, elaborately baffled basins are used. These basins have been

described and their essential features discussed in previous chapters dealing with coagulation and settling in connection with the municipal water purification plants at St. Louis, Mo., New Orleans, La., and Columbus, Ohio. In smaller plants, chiefly for industrial purposes, two types of water-softening apparatus are

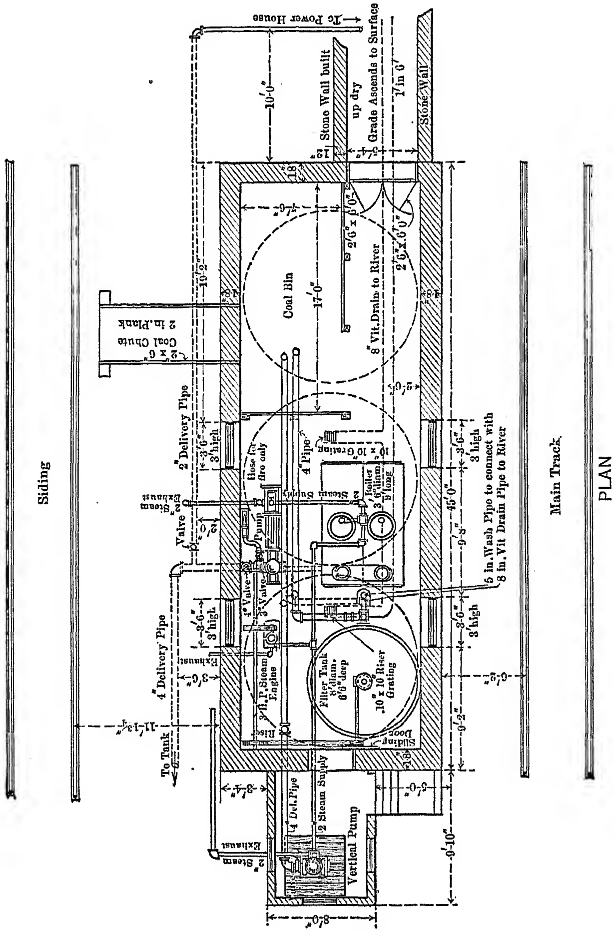

FIG. 134.—Intermittent water softener for railroad water station, plan.

found, namely, those that are intermittent and those that are continuous.

Intermittent Water Softeners.—In apparatus of this type the water to be treated is pumped into tanks, treated with the chemical reagents, sometimes agitated mechanically to facilitate the

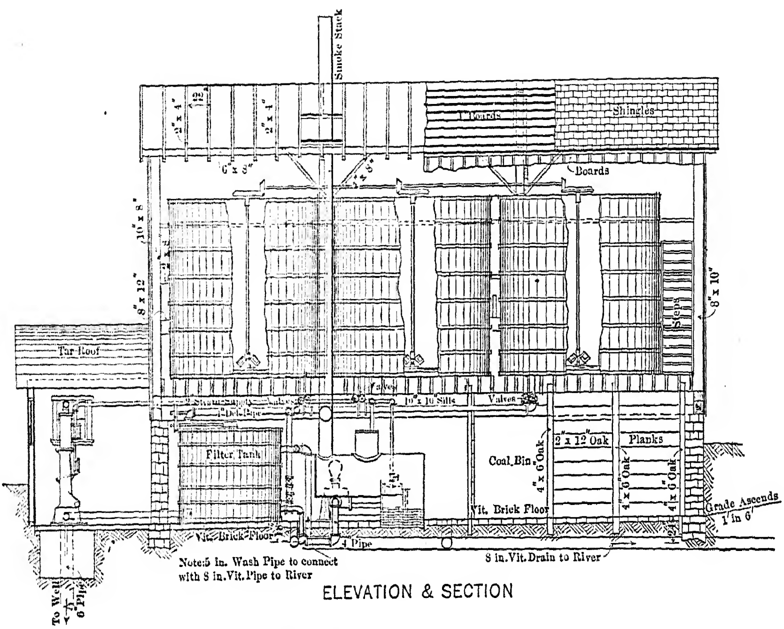

FIG. 135.—Intermittent water softener for railroad water station, elevation and section.

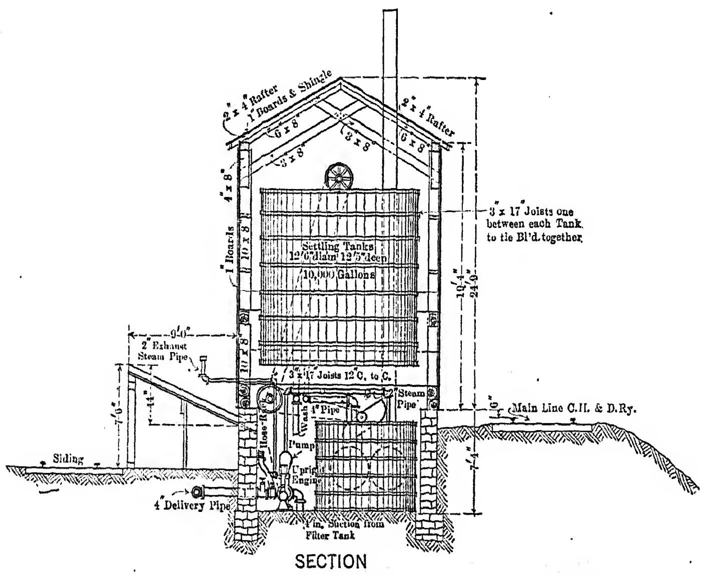

FIG. 136.—Intermittent water softener for railroad water station, section.

chemical reactions, and finally permitted to stand quiescent until the precipitate is deposited. As from 4 to 6 hr. must be allowed for settling, and as to this must be added the time of filling, emptying and cleaning the tank, considerable tank capacity is required. The chemical solutions are prepared at the ground level of the tanks, and are pumped into the latter; or the solid chemicals are hoisted to the top of the tank and dissolved and applied at this point. Agitation of the treated water is effected by paddles turned by an electric motor or water motor, or by compressed air forced in through a grid of perforated pipes on the bottom of the tank.

The softened water is usually withdrawn from the surface by means of a floating pipe. While one tank is being drawn down, another must be filled, treated with chemicals, and allowed to stand for a period sufficient to produce a fairly clear water, so that it may be ready to supply softened water when the first tank has been emptied.

Continuous Water Softeners.—The intermittent system requires considerable space for the tanks, and must receive quite a little attention if properly operated. Continuous water softeners have been designed, therefore, to meet these objections. They are all more or less similar in their general type of construction, and meet the demand for economy of space and automatic operation. Their chief differences are found in the special devices used to measure the water and the applied chemical solutions, and in the means employed to facilitate the deposition of the precipitate in the water which is moving slowly but continuously through the compartments of the softener.

The water enters softeners of this type generally at the top of a tall tank (Fig. 137). As it discharges, it is made to turn a waterwheel, the power from which is utilized to operate agitators in the softening and chemical tanks, and to perform other mechanical work, such as the operation of pumps for handling and preparing the chemical solutions.

The devices used for proportioning the chemical solutions to the flow of water are varied in design. In general, they consist of weirs or orifices over or through which the flow of the chemical solution is made proportional to the flow of hard water into the tank. This is usually effected by a float in the hard-water receiving box. The float rising and falling with the fluctuations in flow operates cutoffs in weirs under constant head, or moves

orifices up or down in order to maintain heads that vary but still are proportional to the head over the hard-water orifice.

The water after receiving its chemical reagents passes into the mixing compartment, and flows downward. This compartment is often made cone-shaped, so that the velocity of the flow will

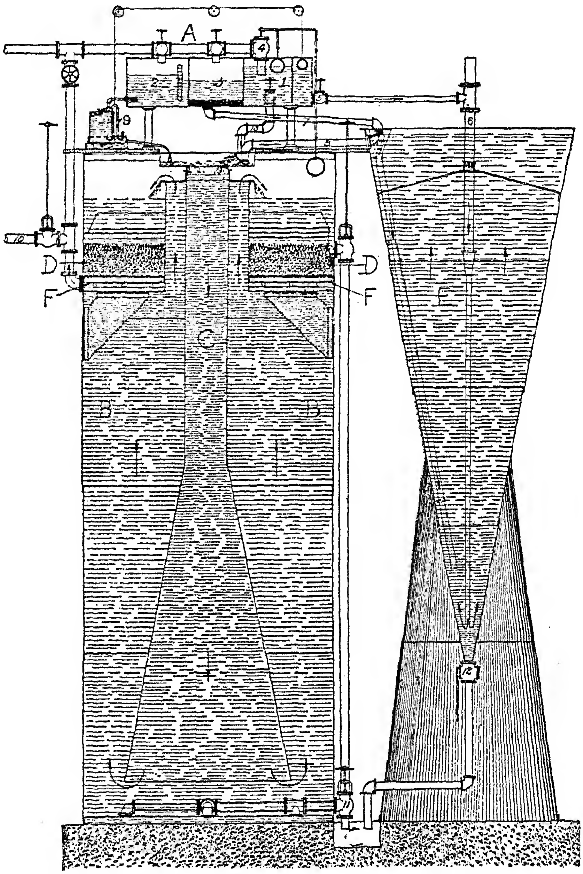

FIG. 137.—Reisert automatic water softener.

lessen as it nears the bottom of the tank. In others it is purely a mixing compartment, being provided with mechanical agitators in the form of stirrers.

After leaving the mixing or central compartment of the softener, the water turns and rises slowly in the annular space around the mixing compartment. Baffling, consisting of inclined per-

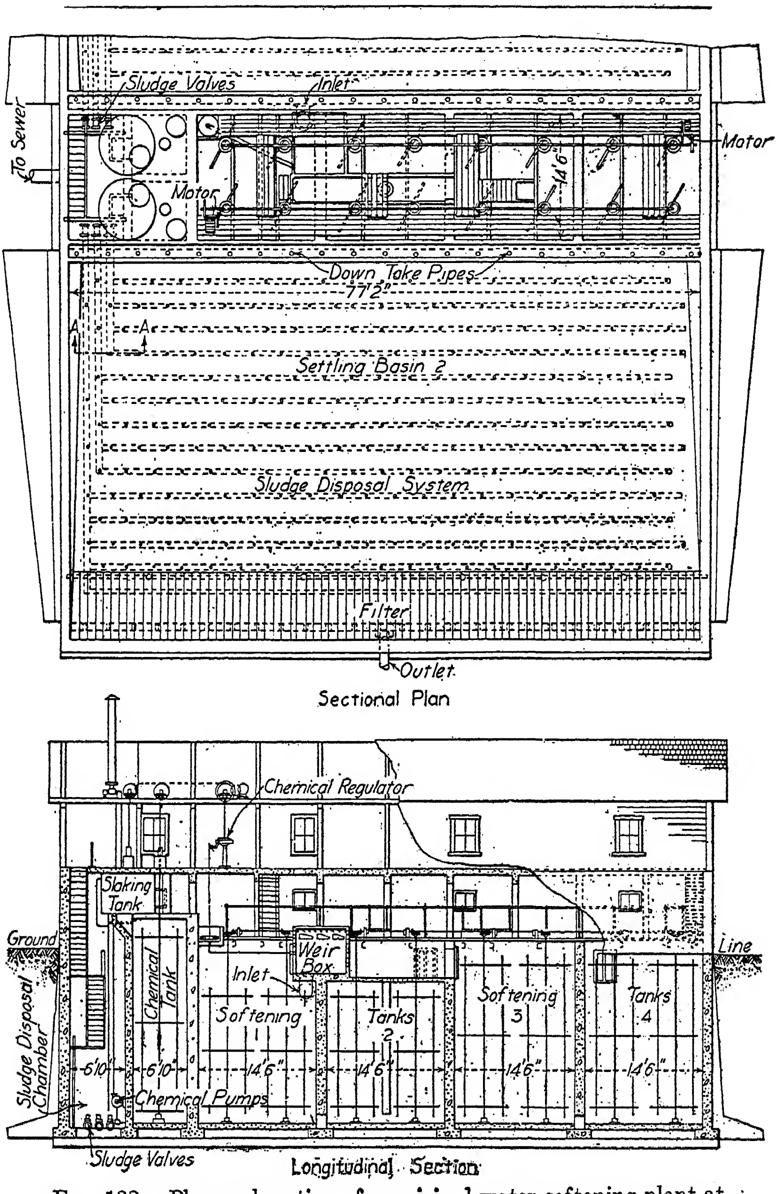

FIG. 138.—Plan and section of municipal water-softening plant at Owensboro, Ky.

forated plates, is sometimes used to facilitate the deposition of the sediment. In other apparatus more space is provided in

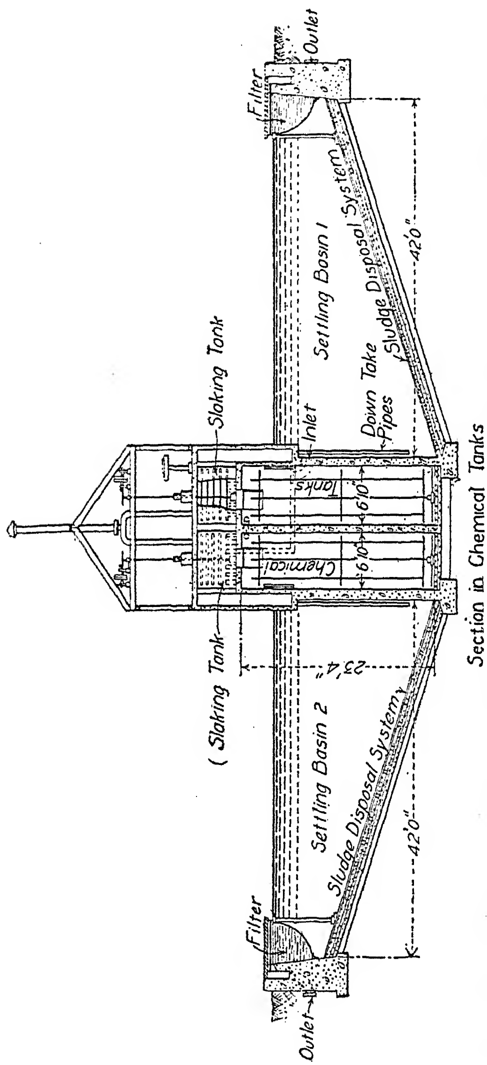

Fig. 139.—Section through settling basins and mixing tanks of water-softening plant at Owensboro, Ky.

this settling chamber, so as to maintain lower velocities, but no baffles are used.

For the final clarification of the softened water filters (Figs. 138 and 139) are generally used. These are frequently made of

wood fiber, filtration being upward through a somewhat shallow layer placed between perforated plates. Filters of sand and of gravel are also used for the purpose of clarifying the water. Rapid sand filters of the usual type are employed in large plants that are softening water for municipal purposes. Sand and gravel filters, as well as those of fiber, must have their filtering material renewed from time to time, since the gradual accumulation of sediment within the beds finally renders them useless. In the case of wood-fiber filters, the fiber is thrown away after it becomes fouled. Sand filters may be partially cleaned by the usual methods of washing with a reverse flow of clean water; but in time this method produces little effect on account of the cementing action of the deposited colloidal precipitates on the grains of sand. Cleaning the sand with hydrochloric acid, whereby the lime and magnesium compounds are dissolved and washed out, is sometimes resorted to. Usually a renewal of the sand is a cheaper method of reconstructing the filter bed, than to attempt to clean the old sand, unless very large quantities of sand are to be replaced.

Messrs. Hoover and Scott[1] state that the sand of the Columbus, Ohio, filters is composed practically of 2 parts of scale and 1 part of sand. The sand has become so badly coated that the effective size has increased from 0.41 to 0.62 mm. These sand grains cement themselves together, forming hard lumps often as large as bushel baskets, and so hard that they must be removed, from the bed to be broken up. It is necessary at this plant to shovel the sand from one bay of the filter to another about twice a year, and requires the labor of five men about 2 weeks' time.

Filtration through sand of a water that has been subjected to the softening process will still farther reduce the residual hardness. A 30-in. sand layer at the Columbus, Ohio, plant will reduce the hardness on an average of 20 per cent., when the raw water is at its maximum hardness. Experimentally it was shown that a 40 per cent. reduction in the hardness could be produced with a bed of sand 6 ft. deep. The phenomenon is one of absorption through the agency of physical forces, and is not a chemical one.

Zeolite Filters.—The practical application of the principles involved in the softening of hard waters by natural or artificial zeolites is effected in ordinary filter tanks of the pressure or

[1] *Monthly Bulletin of the Ohio State Board of Health,* December, 1914.

gravity type, and in which the zeolite takes the place of the usual filter bed. The zeolite, however, does not act as a filtering medium, and turbid waters must first be filtered in order to obtain the best results. Distributing layers of gravel above and below the zeolite layer are sometimes used. In one of the cuts shown (Figs. 140–141) the upper gravel layer rests upon a perforated plate. Between the bottom of the plate supporting the upper gravel layer and the top of the zeolite is an open space. The zeolite, which may range in thickness from 20 to 40 in., rests upon a gravel layer, which in turn is supported on another per-

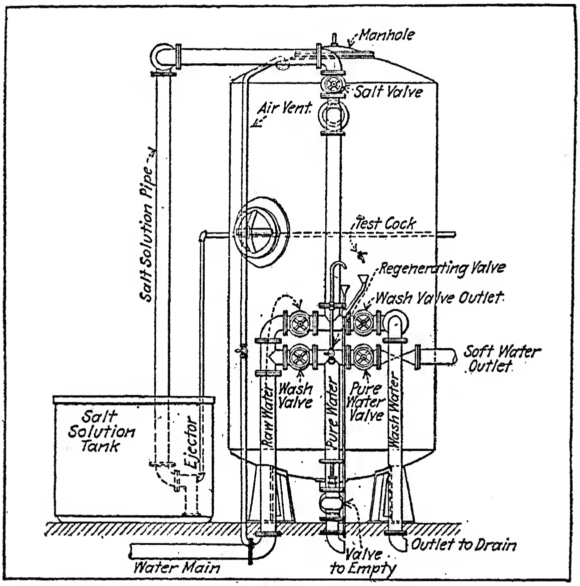

FIG. 140.—Zeolite water softener of the pressure type.

forated plate. Suitable piping, valves and manholes must, of course, be provided for operating the softener.

Brine solution tanks are necessary in this process, since the zeolite is regenerated with a 10 per cent. sodium chloride solution. These tanks are sometimes placed above the softening tanks, and sometimes at the same level as the softener, in which latter case the brine solution must be pumped or ejected into the softener. Where the plant must be operated constantly, all apparatus must be virtually in duplicate, since the regenerative period is practically as long as the operating period.

The rate at which the water passes through the zeolite depends upon its depth and on the hardness of the water. Time must be allowed for the penetration of the water into the interior of the zeolite grains. Wickware[1] states the usual rate of flow through the bed is from 10 to 16 ft. per hour. The extreme limits of speed are for water containing 0.01 per cent. of lime, approximately 27 ft. per hour; with 0.02 per cent., 16 ft.; and with 0.03

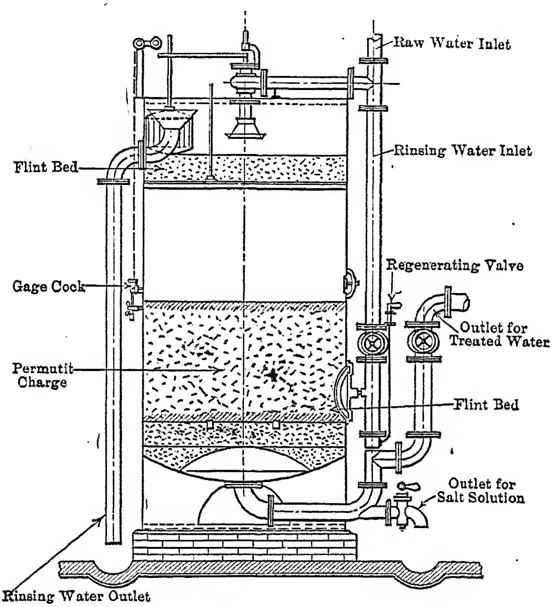

FIG. 141.—Zeolite water softener of the gravity type.

per cent., 10 ft. per hour. Hoover[2] and Scott found the most efficient rates for a water having a total hardness of 392 parts per million to be 55,000,000 gal. per acre per day for a gross rate, and 25,000,000 gal. per acre per day for a net rate. This gross rate (55,000,000 gal. per acre per day) is practically equal to the passage of the water through the bed at a rate of 7 ft. per hour. The water experimented with by these investigators had a hardness in terms of calcium carbonate of 0.0392 per cent.

[1] FRANCIS G. WICKWARE: "European Power Plant Practice." *Practical Engineer*, June 1, 1912.

[2] C. P. HOOVER and R. D. SCOTT: "Water Softening by the Permutit Process." *Ohio Public Health Journal*, August, 1915.

This rate would appear to be in general agreement with the optimum rates stated by Wickware.

Mr. R. N. Kinniard (*Engineering Record,* Dec. 25, 1915) experimented with a natural zeolite and found it to act similarly to the artificial zeolite prepared by Gans. Dr. Edward Bartow has also made experiments with this same material and obtained

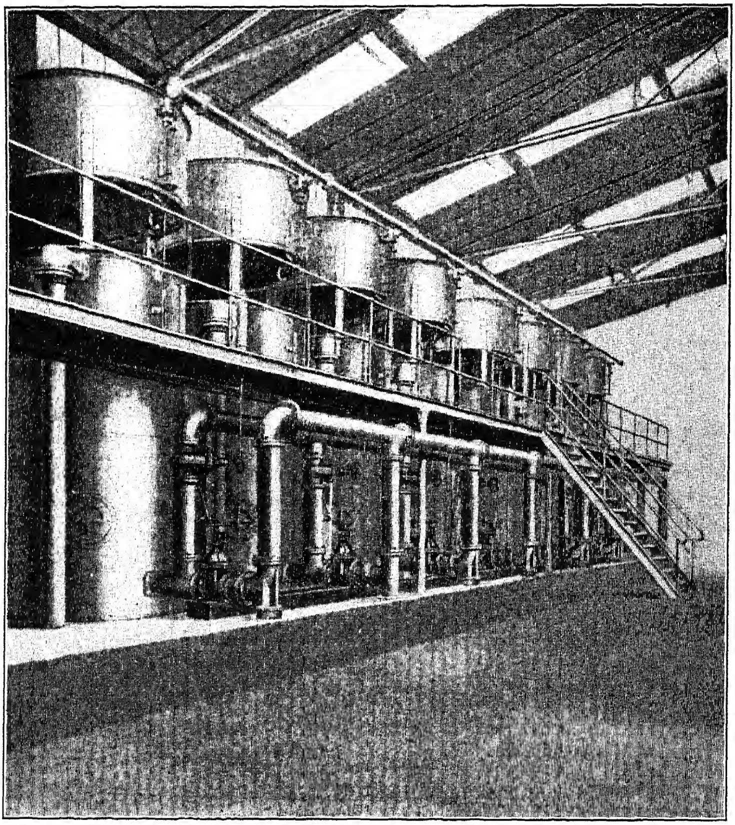

FIG. 142.—A Permutit (zeolite) water-softening plant in Germany.

results similar to those obtained by Mr. Kinniard. In these experiments rates of filtration of 2 gal. per square foot per minute (approximately 16 ft. per hour) gave good results on a water having a carbonate hardness of about 300 parts per million (0.03 per cent.).

The amount of salt required to regenerate artificial zeolite per 1,000 gal. of water treated for each 100 parts per million of hard-

ness was found by Hoover and Scott to be 5.92 lb. Kinniard's figures differ somewhat from these as he was able with the natural zeolite to regenerate with 3⅓ lb. of salt per 1,000 gal. of water treated for each 100 parts of hardness (Fig. 142).

Practical Results Obtained in Water Softening.—The practical benefits derived from a well-softened water used as a public supply lay chiefly in the reduction of the amount of soap used, and in the diminished cost for operating apparatus for the production of hot water and steam. For domestic purposes in bathing, in laundry work and in culinary operations, a soft water is much to be preferred to a hard water. Certain industries, such as paper making, tanning, dyeing and bleaching are able to produce a better product with soft than with hard waters. Mr. George C. Whipple has brought out in a striking manner in his book on "The Value of Pure Water," the depreciated value of a water due to its hardness. The following data are taken from one of the tables in the above-mentioned book.

State	City	Source of supply	Total hardness, parts per million	Depreciation per million gallons
Maine.........	Waterville	Messalonskee River	15	$1.50
Maine........	Augusta	Kennebec River	20	2.00
Massachusetts.	Cambridge	Storage Reservoir	33	3.30
New York.....	Albany	Hudson River	64	6.40
Pennsylvania..	Philadelphia	Schuylkill River	179	17.90
Ohio..........	Toledo	Maumee River	200	20.00
Ohio..........	Columbus	Scioto River	335	33.50

At Winnipeg, Manitoba, Mr. Whipple states that the hardness of the water supply before treatment was 580 parts per million. By softening the hardness was reduced 387 parts per million, or its value was increased $38.70 per million gallons. At Oberlin, Ohio, the hardness of the raw water was 170 parts per million, and by softening it was reduced to 48 parts per million, thereby increasing its value $12.20 per million gallons. These increased values relate only to estimates based upon the greater worth of the water for domestic purposes. If industrial uses were also considered, the value of the softened water would be still further increased.

A summary of certain analytical data obtained in the operation

of the water-softening plant at Columbus,[1] Ohio, is of interest in showing the practical results of softening.

Year	Chemicals applied, grains per gallon			Total hardness, parts per million			Percentage reduction in hardness
	Lime	Soda ash	Coagulant	River water	Settled water	Filtered water	
1909	7.9	3 8	1.76	253	105	93	63.2
1910	7.6	5 3	1 02	270	95	85	68 5
1911	7.5	4.3	1.57	245	90	84	65 7
1912	7 0	3.4	1.90	222	90	80	63.9
1913	7.6	6.0	1 50	271	100	88	67.5

It is worthy of comment that 0.5 to 1.0 grain per gallon of the coagulant (sulphate of aluminum) applied to the softened settled water at Columbus reduces the temporary hardness from 20 to 30 parts per million, whereas the theoretical reduction should be only 8 parts per million. This is accounted for by the precipitation of the basic carbonate of magnesia, which, in its colloidal condition, is dragged down by the aluminum hydroxide produced by the decomposition of the alum.

Hard waters that are colored with vegetable stain are sometimes encountered. The water treated by the Grand Rapids,[2] Mich., plant is of this character. The total hardness of the Grand Rapids' water supply averages about 217 parts per million, and ranges from 104 to 288 parts per million. This is accompanied by an average color of 32 parts per million, ranging from 16 to 55 parts per million. The turbidity ranges from 5 to 135 parts and averages 20 parts per million. It was found that with a color of 30 or 40 parts, from 3 to 4½ grains of alum were required to reduce the color to 10 parts per million. This large amount of alum increased the permanent hardness of the water, beside adding materially to the expense of treatment.

By adding sufficient lime to neutralize all of the bicarbonates present, it was found that a water could be produced with a color of 10 parts or less, and with a total hardness ranging from 88 to 100 parts per million. The alum was, therefore, reduced to ¼ grain per gallon, and the cost of treatment was also lessened.

[1] *Eng. Record,* Apr. 11, 1914.
[2] *Eng. Record,* Sept. 13, 1913.

Mr. S. A. Greeley[1] has recently given some operating data on municipal water-softening plants, which are of interest in this connection.

Plant	Average hardness of raw water	Population, 1910	Quantities		Period for reaction and settling based on actual quantities, hours	Remarks on after-precipitation
			Rated m.g d	Actual m.g.d		
St. Louis, Mo. . .	170	687,029	160	100	40–60[2]	No trouble since filters have been in operation.
Columbus, Ohio.	300	181,511	30	18 4	20 6	
Grand Rapids, Mich.	235	112,171	20	14 0	3 to 3 5	Requires careful operation and mixing. Alum used.
McKeesport, Pa ...	100–600	42,694	10	3 6	7½–20	Incrustation of filter sand. No deposits in mains
Owensboro, Ky.. ..	300	16,011	3	1 5	48	Some precipitate. Not enough to cause trouble.
Oberlin, Ohio.......	300	4,365	0.75	0.28	48	Incrustation on filter sand.
Daytona, Fla.....	360	3,082	0 30	0.17	4–18	A little trouble when settling period is short.
Hinsdale, Ill	360	2,451	1.00	0 30	20–40	
Port Tampa, Fla .	640	1,343	0.24	....	12[3]	

Results Obtained in Softening with Zeolites.—It will probably have been noted that in softening waters with lime and soda ash, the residual hardness in the treated water is still considerable. By passing a hard water through a zeolite bed, it becomes possible to reduce its hardness to zero. This is the only practical method of complete softening known. An equivalent of sodium carbonate is always left in waters softened by this process.

Hoover and Scott in experimenting with the hard Scioto River water and a "permutit" (artificial zeolite) filter obtained the following results.

These investigators regard a water containing so large a quantity of sodium carbonate as undesirable for a public water supply. They made further experiments by filtering the water through the "permutit" after having first reduced the total hardness with

[1] *Jour. Am. Water-works Assn.*, March, 1916.

[2] Based on rated capacity.

[3] Depends on basins in service.

	Raw water	Softened water
	Parts per million[1]	
Total alkalinity	180	185
Bicarbonate alkalinity	180	147
Normal carbonate alkalinity	0	38
Incrustants (permanent hardness)	200	Negative[2]
Total hardness	380	0

lime and soda ash to about 91 parts per million, the object being to make use of the "permutit" for a finishing process. A sodium carbonate alkalinity in the water passed through the "permutit," equal to 62 to 105 parts per million, was found, depending upon whether nearly exhausted or fresh "permutit" was used. These quantities are still large and are somewhat objectionable in a water to be used for boiler purposes, on account of the foaming which the sodium carbonate would possibly produce. Their conclusions[3] as to the merits of the "permutit" are as follows:

"*Advantages of the Permutit Process.*—(1) It is the only practical process, known to the writers, by which water of a zero hardness can be produced on a large scale.

"2. Only one chemical is needed (common salt).

"3. Variations in the hardness of the raw water are automatically taken care of.

"4. There is no sludge to be removed.

"*Disadvantages.*—(1) Cost of operation is higher than with lime and soda ash.

"2. Water to be softened must be perfectly clear, for if it contains turbidity the pores of the permutit become clogged.

"3. The permutit-softened water contains residual sodium bicarbonate, and if used in boilers may cause trouble by foaming."

Bacterial Effects of Water Softening.—Bacterial removal by coagulation and precipitation is usually very efficient in water-softening plants. This is commonly due to the gelatinous character of the magnesium hydroxide produced when enough caustic lime is added to form this precipitate, and to the causticity of the water itself. The concentration of hydroxyl ions is sufficient to

[1] Expressed in terms of $CaCO_3$.

[2] Excess of sodium carbonate, 182 parts.

[3] *Ohio Public Health Journal*, August, 1915.

effect a coagulation of the positively charged clay particles, which form masses that settle rapidly.

Beside mechanically assisting in the removal of the bacteria, the causticity of the water appears to actually kill intestinal bacteria, thus acting as a germicide. This may not be a specific toxic effect, but rather a secondary effect arising from the removal of the carbonic acid. It is claimed that a water in which the free and half-bound carbonic acid are removed, but which is not caustic, will act similarly toward intestinal organisms.

In the treatment of the water supply of London, England, Dr. A. C. Houston, Director of Water Examination of the Metropolitan Water Board, made some studies which led to the conclusion that treatment of the water with quicklime in an amount sufficient to cause an excess of calcium oxide equal to 0.007 per cent. caused the death of B. coli in from 5 to 24 hr. As a result of this work, Dr. Houston proposed to treat about 75 per cent. of the water supply with a slight excess of lime, and this excess to be neutralized by mixing with the remaining 25 per cent. of the water, previously purified by long storage, or by the use of some other disinfectant. In this connection Mr. W. H. Dittoe, Chief Engineer of the Ohio State Board of Health,[1] states, that:

"While this method of disinfection of water has been carefully studied, several important features of the treatment remain to be demonstrated. Its bacterial efficiency has been shown, but its universal applicability has not been proven. The principal objection which has been advanced relates to the cost of the treatment necessary to secure disinfection. In this connection, however, due credit should be given to the beneficial softening effect also produced by the treatment. Its use as an available method for purification of water supplies for private use has not been demonstrated."

Mr. Russel D. Scott,[2] assistant bacteriologist of the Ohio State Board of Health, has made some interesting studies on the bacteriological effects of water softening as carried out on the Scioto River water at Columbus, Ohio. His conclusions are as follows:

"1. The bacteriological character of the Scioto River water during periods of long storage varies from that during flood stages, in that: (*a*) the number of bacteria is much less; (*b*) the 37° agar count is a much higher proportion of the 20° count; (*c*) of the B. coli group, the saccharose fermenting type are the most numerous.

[1] *Ohio Public Health Journal*, August, 1915.

[2] *Ohio Public Health Journal*, June, 1915.

"2. The lime softened water varies bacteriologically from the raw water, in that: (*a*) the number of bacteria is much less; (*b*) the ratio of the number of B. coli to the total number of bacteria is less; (*c*) the saccharose-fermenting types of B. coli are much more numerous; (*d*) the proportion of dextrose fermenters which do not ferment lactose is much greater; (*e*) the ratio of the 37° to the 20° count is greater; (*f*) the ratio of the gelatine to the agar count is very much higher.

"3. The proportion of organisms not fermenting dulcite is not affected either by storage or by lime-treatment. This is also true of those that do not form indol.

"4. The members of the B. coli group form somewhat less than 4 per cent. of the total number of bacteria at 37°C. This ratio is, however, quite irregular.

"5. The proportion of organisms in the river water which ferment dextrose but not lactose is not affected by storage."

Annual Cost of Operation and First Cost of Water-softening Plants.—Mr. S. A. Greeley has recently compiled some data upon the cost of operation and the first cost of water-softening plants. The following tables are taken from his article on "Water-softening Practice" in the March number of the *Journal of the American Water-works Association* (vol. 3, No. 1, 1916).

Plant	Population, 1910	Actual quantity, millions of gallons per year	Average pounds chemicals used per million gallons					Operators required, total	Cost of operation per million gallons
			Lime	Soda ash	Iron sulp.	Alum sulp.	Hypochlorite of lime		
St Louis, Mo.......	687,029	34,656	800	...	415	.....	1.6	35	$4.56[1]
Columbus, Ohio.....	181,511	6,716	1,384	1,054	...	260	4.0	....	17.46
Grand Rapids, Mich	112,171	4,506	1,250		...	120	...	16	11.34
McKeesport, Pa. ...	42,694	1,314	100 3,500	100 6,000	50 200	75 400	...	....	27.57
Owensboro, Ky	16,011	547	2,000	75	...	.....	1.0	2[2]	10.05
Oberlin, Ohio......	4,365	102	2,045	735	...	.. ..	...	1[2]	13.49
Daytona, Fla.	3,082	62	2,910	250	...	.....	...	1[2]	23.80
Hinsdale, Ill........	2,451	100	4,091	1,368	...	Small amount	...	1[2]	30.00

[1] Does not include pumping.
[2] Part time of one man.

The hardness of the St. Louis raw water is 170 parts per million. The raw water treated at Columbus has a hardness of about 300 parts, and at Oberlin and Owensboro it averages the same. At Grand Rapids the raw water averages about 235 parts

per million of hardness, and at Daytona and Hinsdale it is 360 parts per million. The raw water at McKeesport ranges from 100 to 600 parts of hardness.

The cost of operation is naturally affected by local conditions other than the hardness of the water. The size of the plant and the cost of the chemicals in different parts of the country, are factors that must be considered in making comparisons of costs.

The first cost of these water-softening plants is shown by the following table, which is also taken from Mr. Greeley's paper.

Plant	Date of erection	Population, 1910	Quantities		Cost of construction		Remarks
			Rated m.g.d.	Actual m.g.d.	Total	Per million gallons rated capacity	
St. Louis, Mo. .	1915	687,029	160.00	100.00	$1,495,000	$9,344	Cost approximate.[1]
Columbus, Ohio.	1906	181,511	30.00	18.40	590,780	19,693	First-class construction.
Grand Rapids, Mich..........	1910	112,171	20.00	14.00	449,569	22,478	First-class construction.
McKeesport, Pa.	1908	42,694	10.00	3.60	250,000	25,000	First-class construction.
Owensboro, Ky.	1906	16,011	3.00	1.50	30,000	10,000	Excelsior filter.
Oberlin, Ohio ..	1903	4,365	0.75	0.28	12,000	13,300	Pressure filter.
Daytona, Fla....	....	3,082	0.30	0.17	4,304	14,347	No filters.
Hinsdale, Ill	1915	2,451	1.00	0.30	18,000	18,000	Filters 18 in. deep.
Port Tampa, Fla	1914	1,343	0.24	....	7,000	29,100	No filters.

[1] Existing work also used, but not included in figures.

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CHAPTER XXXI

THE REMOVAL OF DISSOLVED MINERAL MATTER FROM WATER (CONTINUED)

REMOVAL OF IRON AND MANGANESE

The use of ground waters for public water supplies is common, and undoubtedly possesses certain advantages as well as some disadvantages. Many ground waters are entirely suitable for domestic and industrial purposes, except for the iron and manganese which they contain. Waters which contain less than 0.2 part per million of iron are not objectionable, and even as much as 0.5 part may give little trouble. Iron is generally more easily separated from a water than is manganese.

Iron is commonly present in the ferrous condition, that is as ferrous hydrate, ferrous bicarbonate or ferrous sulphate. Organic matter, also, in some cases plays an important part in holding both iron and manganese in solution. Iron, if present as a bicarbonate, can usually be precipitated from a ground water by aeration, if the content of manganese and organic matter is low.

By aeration the carbon dioxide is removed and the iron is oxidized to the ferric condition, in which form it is quite insoluble in the water. Manganese reacts more slowly with oxygen, and its precipitation requires a much longer period than does the iron. Filtration through sand and gravel after aeration effects the removal of both the iron and manganese in suspension, and doubtless also some that may be in the form of a colloidal suspension. In general it may be said, that the precipitation of iron is interfered with by carbon dioxide, organic matter and manganese.

Weston[1] has noted that the precipitation of aluminum is more difficult than that of iron, but less so than that of manganese. The metal which forms the heaviest oxide is first precipitated. The same investigator found that in three waters containing both

[1] R. S. Weston: "Some Recent Experiences in the Deferrization and Demanganization of Water." *Jour. New England Water-works Assn.*, February, 1914.

iron and manganese, one could not be aerated to saturation without causing some of the iron and color to escape removal; one required complete oxidation to effect a satisfactory removal of manganese; and one demanded long aeration and contact in a trickling filter (which it was necessary at times to operate submerged) to produce satisfactory results. Other things being equal, it is well to remove carbon dioxide as completely as practicable in order to prevent its corrosive effect on the distribution piping.

It is quite evident from what has been stated above that waters containing iron and manganese vary in character, and that methods especially adapted to their peculiar properties must be used to effect purification. The problem is not rendered any easier of solution by the fact that water from the same source is not always constant in its characteristics, and hence may need at times a change in the method of treatment in order to meet these changed conditions. It is probable that much of the phenomena connected with the purification of waters containing iron and manganese can be explained by the quantitative relations existing between the concentrations of the several components at equilibrium.

Iron Removal in Presence of Carbon Dioxide.—When iron is present in a water as the bicarbonate, it is accompanied by a considerable amount of free carbon dioxide. Johnston[1] has pointed out in a recent paper that in any solution containing a carbonate, "there is a readily attained equilibrium between the carbonate ion, $CO_3^{=}$, the bicarbonate ion, HCO_3^{-}, and the carbonic acid, H_2CO_3, and in turn between the carbonic acid and the partial pressure of carbon dioxide above the solution; consequently, these molecular species can coexist only in definite proportions determined by the several equilibrium constants."

By agitation a considerable portion of the CO_2 may be removed from the water. The oxygen dissolved in the water is then able to oxidize the iron, and thereby render it insoluble. More or less absorption of oxygen from the air takes place during the agitation of the water, and the oxidation and resultant precipitation of the iron is thus hastened. The removal of carbon dioxide by adding to the water an alkali, such as lime or soda ash, pro-

[1] John Johnston: "The Determination of Carbonic Acid, Combined and Free in Solution, Particularly in Natural Waters." *Jour. Am. Chem. Soc.*, May, 1916.

duces a like result. Rapid filtration of the aerated and settled water through sand is usually the final step in the process of purification.

As a typical example of waters of this type, Mr. Frank E. Hale[1] gives the following results obtained in treating in the laboratory a Long Island (N. Y.) well water by aeration, and also with lime and with soda ash:

Analysis	Parts per million
Free carbonic acid (CO_2)	31.2
Iron (Fe)	11.2
Hardness ($CaCO_3$)	47 0
Alkalinity ($CaCO_3$)	32.0
Chlorine (Cl)	64.0
Nitrate (N)	0 0
Experimental treatment	
Iron after aeration for ½ hr. and filtered through paper	7.2
Iron after neutralization of carbonic acid with soda ash and filtration through paper	0.3
Iron after neutralization of carbonic acid with calcium hydrate, and filtration through paper	0 1

	Rates of filtration			
	350 gal per minute		650 gal per minute	
	Free CO_2	Iron (Fe)	Free CO_2	Iron (Fe)
	Parts per million			
Raw water	55	5 00	49	5.40
After nine aerators	41	5.00	40	5.40
After eleven aerators and sedimentation tanks (water to filter)	31	3 50	31	4.20
Effluent from filters	19	0.05	22	0.05

N. B.—A rate of discharge of 350 gal. per minute was equal to a rate of 69,000,000 gal. per acre per day, and a discharge of 650 gal. per minute was equal to a rate of 114,000,000 gal. per acre per day.

[1] "Iron Removal by Rapid Sand Filtration." *Jour. Am. Water-works Assn.*, March, 1916.

This same water treated in a purification plant in which aeration was effected by a series of cascades over which the water splashed and foamed before entering a sedimentation tank, gave the foregoing results shown in the preceding table.

The reduction in the quantity of CO_2 as well as of the iron should be particularly noted in these results.

The so-called Anderson revolving purifier, used in a few places in Europe, consists of a rotating drum containing a quantity of iron turnings. The water passes through the drum and is agitated in contact with the iron turnings. By the action of the excess CO_2 more iron is dissolved, and as a result of the agitation more or less CO_2 is also probably removed. Oxidation of the iron to the ferric condition then follows, and by its precipitation

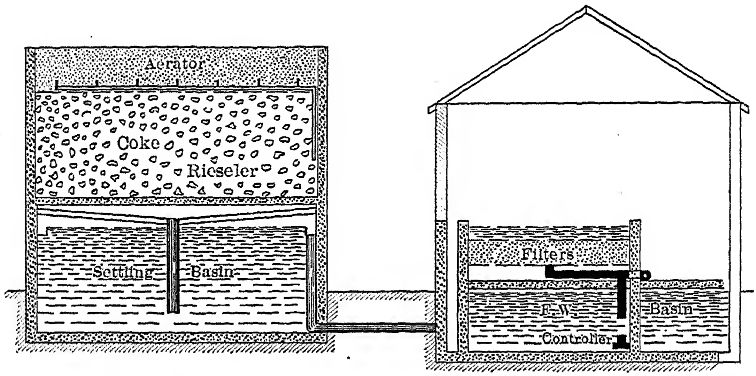

FIG. 143.—Sketch showing arrangement of Cohasset, Mass., deferrization plant.

acts as a coagulant, and assists in precipitating suspended matter and even colloidal suspensions such as vegetable coloring matter.

Iron Removal in the Presence of Organic Matter.—A well water which contained considerable organic matter, carbon dioxide, iron and some manganese, was experimented with by Mr. R. S. Weston at Cohasset, Mass., in 1913 (Fig. 143). The color of the water ranged from 35 to 50 parts per million, the iron from 0.4 to 1.15 parts, the CO_2 from 40 to 56 parts and the manganese from 0.15 to 0.4 part.

By means of an aerator containing 6 in. of stone, a "trickler" filled with coke, subsiding basins and a mechanical filter, the following purification effects were produced experimentally:

	Well water	Effluent from rieseler	Effluent from filter
	Parts per million		
Color	50 00	40.00	35.00
Iron	0 90	0.35	0 28
Carbon dioxide	54 00	8.60	8.00
Dissolved oxygen	2 17	10.39	10.25

NOTE.—"Trickler" or rieseler operated at rate of 50,000,000 gal. per acre per day, and the filter at 100,000,000 gal. per acre per day.

Mr. Weston's comments on the experimental results are as follows:

"It will be readily observed that the bulk of the work was done before filtration. The reason for this is that contact between the water and the rough surfaces of the stones or coke effects the displacement of the carbon dioxide by oxygen; the oxidation of the iron and the manganese; and the removal of the color. The filter acts simply as a strainer. At the beginning of the experiments, the stones were clean, but in less than a month they showed a red coating. This, on analysis, was found to be iron rust mixed with some organic matter. In other words, notwithstanding the fact that the season was most unfavorable, on account of low temperature, for the establishment of contact action, the stones had begun to remove both iron and organic matter to a satisfactory degree."

Iron Removal in Presence of Manganese.—A well water in which manganese interferes with the removal of the iron is found at Middleboro, Mass. Experiments by the Massachusetts State Board of Health in 1911–12, and by Mr. R. S. Weston in 1913, have shown the difficulties connected with purifying this type of water. In Weston's experimental apparatus the water was delivered to a spray aerator, which discharged through the air a distance of about 20 in. to a coke "trickler." The latter was operated at a rate of 75,000,000 gal. per acre per day. For a depth of 6.7 ft., the coke of the "trickler" was not submerged. Contact with submerged coke, however, was obtained in the lower portion of the "trickler." The filter consisted of 26 in. of sand with an effective size of 0.28 mm. It was operated at a rate of 10,000,000 gal. per acre per day.

The results of the operation of these devices on Mar. 27 and Apr. 23, 1913, are given in the following table:

	Well water		Trickler effluent		Basin effluent		Filter effluent	
	Mar. 27	Apr. 23	Mar. 27	Apr. 23	Mar. 27	Apr. 23	Mar. 27	Apr. 23
	Parts per million							
Color	45.00	45.00	40.00	38.00	12.00	10.00	10.00	10.00
Turbidity	3.00	2.00	5.00	5.00	2.00	2.00	1.00	1.00
Iron	1.20	0.90	1.90	1.80	0.35	0.25	0.15	0.20
Manganese	0.75	0.72	0.95	0.90	0.75	0.55	0.38	0.25
Carbon dioxide	44.00	48.00	5.00	5.00	5.50	5.30	5.00	5.00
Dissolved oxygen	3.50	4.21	11.07	11.42	10.83	11.51	10.91	11.26

Middleboro Purification Plant.—The plant designed and built by Messrs. Weston and Sampson[1] to treat the water at Middle-

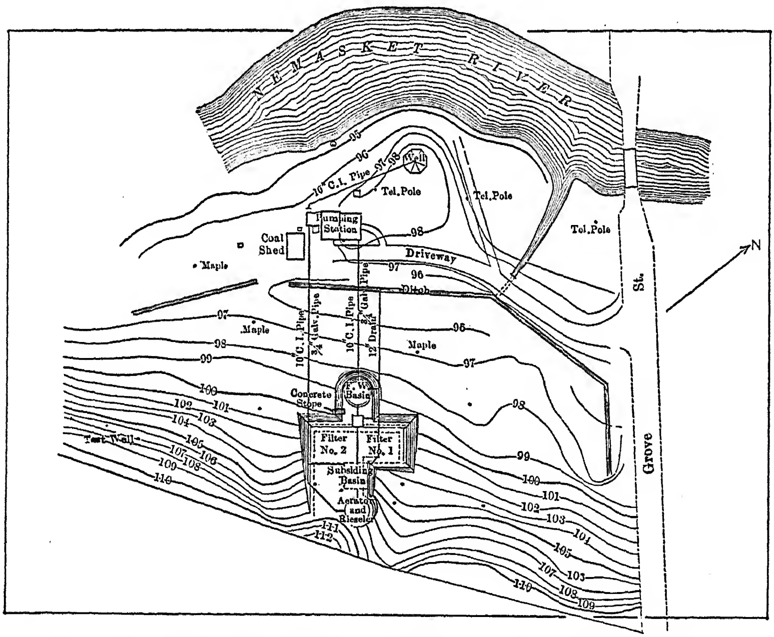

FIG. 144.—Middleboro, Mass., deferrization plant, general plan.

boro, Mass., as a result of the experiments mentioned above, is described in some detail as typical of well-designed plants for treating iron and manganese waters.

The following description, cuts and illustrations are taken

[1] *Jour. New England Water-works Assn.*, vol. 28, No. 1, 1914.

from Mr. Weston's paper, "Some Recent Experiences in the Deferrization and Demanganization of Water" (Fig. 144).

"**General Description of Plant.**—A DeLaval steam-turbine-driven, centrifugal pump, having a capacity of 1,000,000 gal. per day, takes water from the present well and discharges it on top of the pile of coke —a so-called trickler—through a number of sprinkling nozzles. The trickler is built of concrete. It is 30 ft. in diameter, outside measurements, and contains 10 ft. of coarse coke, supported upon a grillwork above the true bottom. The water from the trickler falls upon its true bottom and flows through a pipe into the settling basin.

"The settling basin is built of concrete and has a capacity of 40,000 gal. It is covered with a concrete roof and earth to prevent freezing of the water.

"From the settling basin the water flows into the two compartments of the filter, which has a total area of 0.1 acre. These filters are simply concrete basins with groined roofs, covered like the settling basin with earth to prevent the freezing of the water in winter. On the bottom of the filter are laid tile underdrains. These are covered with 12 in. of graded gravel. Above the latter layer and supported by it is a layer of sand 3 ft. thick.

"From the two filters the water flows into a regulator house in which are located all of the valves and gages for operating and regulating the filters. From the regulator house the water flows into a filtered-water basin which holds 42,000 gal. This is large enough to enable the pumps and filters to operate regularly and to supply the hydrants during fires of an ordinary character. For fire purposes this supply is at the rate of at least 1,000,000 gal. in 24 hr., or twice this amount during 1 hr. The present consumption is about 335,000 gal. per day; and the maximum consumption for any one day has been 873,000 gal.

"Of these various parts the aerator and trickler are alone worthy of special comment, for although the filters embody some special features these may be readily apprehended by inspecting the plates.

"*Aerator and Trickler.*—The aerator consists of a system of piping connecting the discharge from the low-lift pump with thirty-seven 2-in. nipples. On each nipple is screwed a cap drilled with twenty-four $\frac{3}{16}$-in. holes, and pressed to make the upper surface convex and the axes of the holes radial. These sprays discharge upward, throwing the water on the surface of a layer of coarse coke, 10 ft. deep, contained in a concrete tower 28.3 ft. in diameter (Fig. 145).

"The coke is supported upon 2 in. by 6-in. reinforced-concrete beams spaced 2 in. apart, and supported upon a beam 6 in. wide extending around the inside of the wall of the trickler and also upon four cross-walls, 6 in. wide, resting on the floor. The floor slopes to a channel leading to the 18-in. cast-iron pipe, which connects with the subsiding

basins and filters. A shear gate provides for the closing of this outlet, when the trickler may be filled; then the valve may be opened quickly and the excess of accumulation in the coke flushed out through the subsiding basin and into the drain. It is not expected that flushing will be required oftener than once a year.

"*Filters and Appurtenances.*—The two filters have no doors and runways, but special sand boxes have been designed into which the sand removed by scraping is dumped and washed out into bins placed outside the filter. The sand used in the filters had an effective size of 0.31 mm., and a uniformity coefficient of 1.8. It was natural bank sand which did not require washing (Figs. 146 and 147).

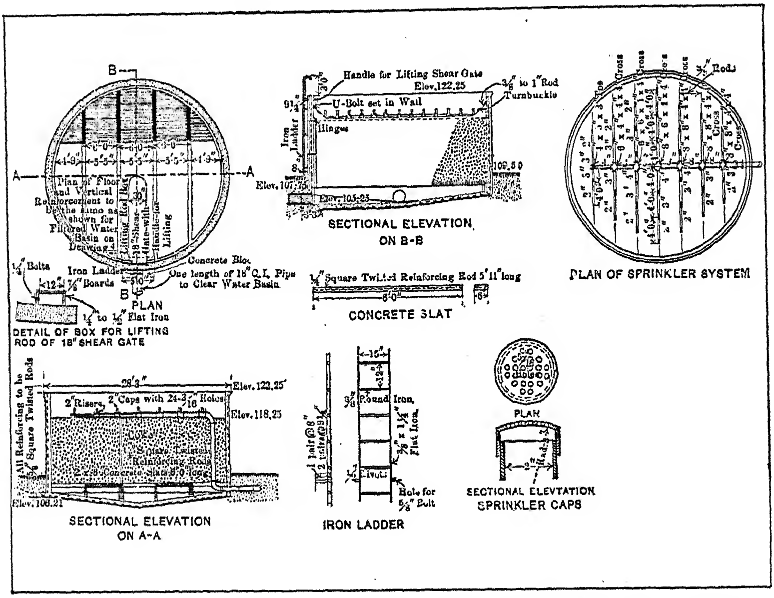

FIG. 145.—Coke "rieseler" or trickler with aerator, Middleboro, Mass.

"The regulating devices are of the orifice type, and indicating rate and loss of head gages are provided for each filter.

"*Cost.*—The complete plant has cost about $18,000, including engineering.

"*Results of Operation.*—The filters were started on Sept. 26, 1913. They were raked twice and scraped once prior to Jan. 12, 1914. The yield for this first run was 40,000,000 gal., or at the rate of 400,000,000 gal. per acre. The average results for this first run, between the dates above mentioned, are as follows:"

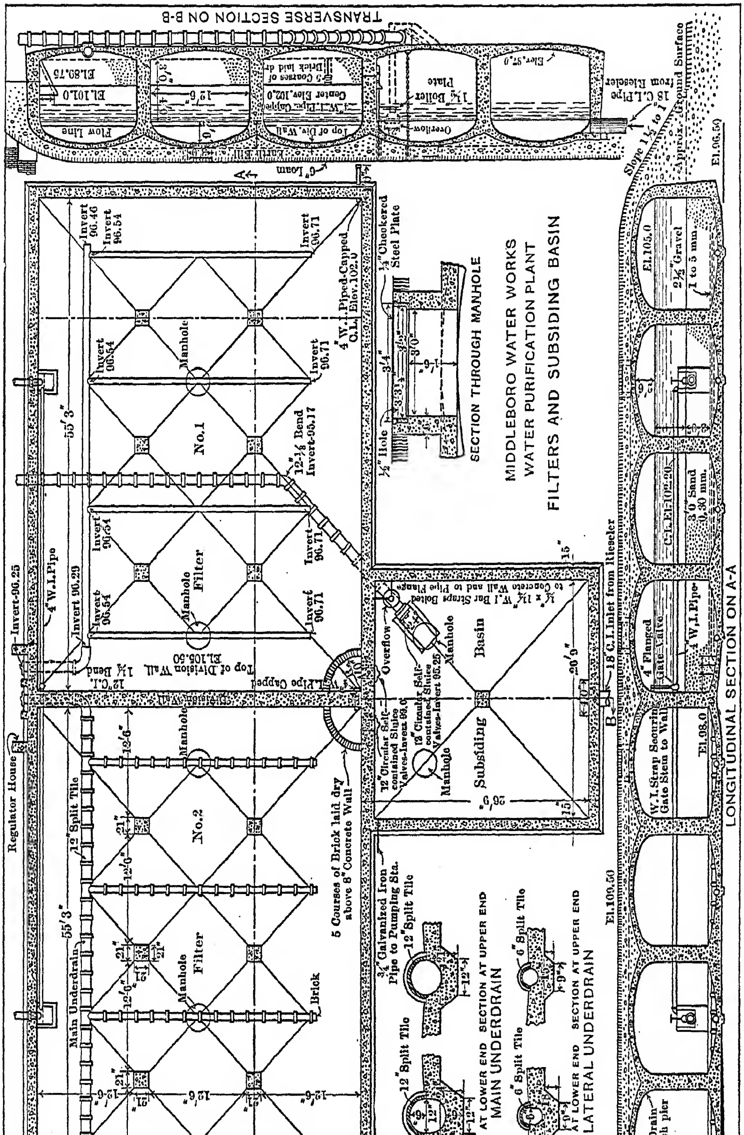

FIG. 146.—Filters and subsiding basins at Middleboro, Mass., plant.

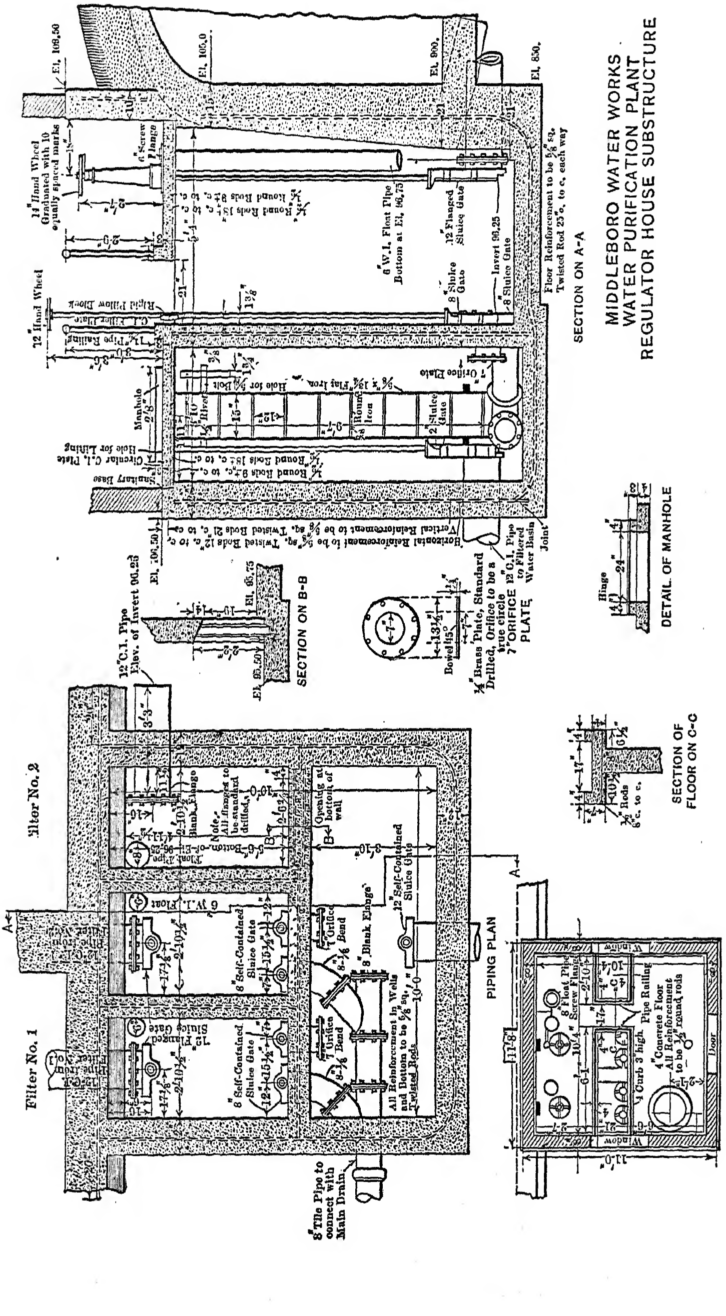

FIG. 147.—Regulator house and substructure of Middleboro, Mass., plant.

	Well water	Settling basin effluent	Filter effluent
	Parts per million		
Color	48.00	22 00	5 00
Turbidity	5.00	3.00	1.00
Iron	1 62	0 46	0 17
Manganese	0 67	0 36	0 27
Hardness[1]	27 30	....	23.40
Oxygen consumed	1.79	1.53	1.05
Carbon dioxide[1]	41 00	4 20	4.60
Dissolved oxygen[1]	2 95	10 20	9 55

[1] One determination.

Manganese in Ground Waters.—It is usually found that waters which contain considerable amounts of iron will also contain manganese. In some instances, however, large quantities of manganese are found associated with quite small amounts of iron. A case in point is the water supply of Dresden, Germany, which contains from 0.6 to 1.0 part per million of manganese, but only about 0.1 part of iron. Where manganese occurs as the sulphate, its removal is difficult, the reason being that manganese salts of active acids are not oxidized by the air, as are the corresponding salts of iron.

Manganese like iron is especially troublesome in waters that are to be used in laundries, bleacheries, dye houses and paper mills, causing stains or dulling the effect of the bleaching or dyeing. Drinking water, which contains those amounts of manganese that are usually found in water supplies, appears to have no injurious effect. The growth of the "iron bacteria" in distribution pipe lines is not an uncommon feature of waters that contain iron and manganese.

In a great many European water supplies (Figs. 148 and 149), the occurrence of manganese is common, and the amounts found range from 0.1 to 6.5 parts per million. Iron, as stated above, is usually found associated with the manganese. The experience of the City of Breslau on the Oder River with the pollution of its ground-water supply in 1906 by iron and manganese, as reported by Mr. S. B. Applebaum (*Journal Industrial and Engineering Chemistry*, February, 1916) is of interest, because of the unusual circumstances.

The city had been using since 1871 Oder River water filtered through slow sand filters. As it was believed that a ground-water supply would be safer, it was decided to obtain water from wells, although it was known that the water would probably contain iron. A series of 300 6-in. wells were bored about 60 ft. apart, and the water pumped from them was purified of its iron and manganese in "tricklers" and sand filters. The yield of the wells proved to be only about 10,000,000 gal. of water daily, whereas 15,000,000 gal. had been expected. As the pumping of the wells continued, the iron content of the water rose from 6 to 20 parts per million, but the purification plant was able to remove

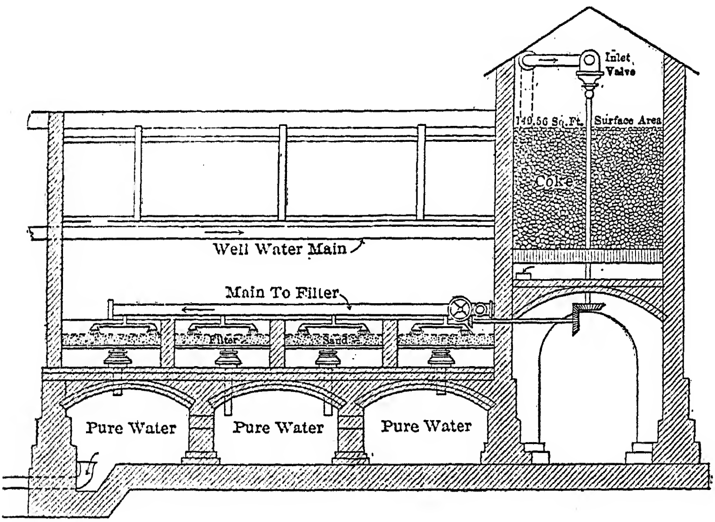

FIG. 148.—Deferrization plant at Hamburg, Germany.

it. Early in 1906 after a season of drought, it was noted that the water table fell steadily. On Mar. 28, 1906, a flood in the Oder River overflowed the ground on which the wells were located. The chemical composition of the water changed over night, the total solids trebled, and the iron content of the water rose to 100 parts per million, and the manganese to 50 parts per million. The water was also found to be slightly acid.

The purification plant was unable to purify the water. After a while most of the wells again began to yield a normal water, but the remainder were finally abandoned. This phenomenon was repeated again in September of this same year.

The probable explanation of this sudden pollution of the well waters is that the level of the ground water, being drawn so low, permitted the oxygen of the air to oxidize the iron and manganese sulphides in the soil to sulphates. The decomposition of organic matter furnished the hydrogen sulphide which had originally converted the iron and manganese to sulphides. When the flood in the Oder River came, it washed the sulphates, formed as described above, into the wells.

Dr. H. Luhrig undertook experimental work to determine the best method of removing the iron and manganese from the water at Breslau, and also from the well waters of Glogau's proposed supply on the opposite side of the Oder River. The general outline of the experiments and the conclusions reached by Luhrig, as reported by Applebaum, are as follows:

"*Aeration and Filtration.*—This method is successful if considerable iron is present with the manganese, and where both are present as bicarbonates. Aeration may be effected in tricklers or under pressure in a closed shell. This method will not remove manganese in any appreciable amount if the content of iron is low, nor where the manganese is present as the sulphate; and under the most favorable conditions the manganese is not entirely removed.

"*Aeration and Filtration, Addition of Lime, Settlement and a Second Filtration.*—The greater part of the iron and manganese were removed by aeration and filtration. A slight excess of lime water (2 to 4 per cent.) was then added. Settlement in basins followed by filtration through sand, completed the process. This method is a practical one. Its disadvantages at Breslau were found to be the care required in applying the lime water to a water constantly varying in its composition, and the disagreeable taste imparted to the water by the excess of lime applied. This taste was eliminated by adding alum, free CO_2, or by mixing the effluent with filtered river water.

"*Addition of a Permanganate.*—Oxidation of the manganese was quite successful. The cost for removing 8 parts per million of manganese sulphate was $6 per million gallons, and was regarded as low. One of the objections to the use of a permanganate is that one of the products of the oxidation is sulphuric acid, which must be neutralized. Over- or underdosing with this reagent may give trouble.

"*Ozone or Electrolysis.*—Both methods are able to produce satisfactory results, ozone being more effective as an oxidizing agent than a permanganate. The oxides produced, however, are very finely divided, and filters must be operated at low rates in order to successfully remove them.

"*Filtration through "Manganese Permutit."*—This reagent like the

"sodium permutit" described in a preceding chapter, is an artificial zeolite. The sodium "permutit" is treated with a dilute solution of manganous chloride, and forms "manganese permutit" by an exchange of the manganese for the sodium in the zeolite. The "manganese permutit" is then oxidized by a 2 to 3 per cent. solution of sodium permanganate. It is presumed that higher oxides of manganese coat the grains of the reformed "sodium permutit." If the formula of the "permutit" is $2SiO_2 \cdot Al_2O_3 \cdot Na_2O$, and is denoted by the formula Na_2O Pmt, the reactions may be shown as follows:

1. $Na_2OPmt + MnCl_2 \quad = MnOPmt + 2NaCl.$
2. $MnOPmt + 2NaMnO_4 = Na_2OPmt + MnO{,}Mn_2O_7.$

"The manganese oxides exert a strong oxidizing action on the manganese dissolved in the water, and the "sodium permutit" acts presumably only as a carrier. By passing the water to be treated through a bed of this artificial zeolite, prepared as above described, the following reaction probably ensues:

1. $Na_2OPmt + MnO,\ Mn_2O_7 + 2Mn(HCO_3)_2 = Na_2OPmt + 5MnO_2 + 4CO_2 + 2H_2O.$

"When the higher oxides of manganese on the surface of the "sodium permutit" are gone, the filter bed becomes inactive, and must be regenerated with sodium permanganate. About 77,200 gal. of water were passed through the bed per pound of sodium permanganate used for regeneration; the rate of filtration being 9.2 gal. per square foot of bed per minute. About 1 per cent. of wash water was used to wash out the deposited oxides of iron and manganese."

The experimental "manganese permutit" plant at Glogau consisted of closed filters 3.5 ft. in diameter and 6 ft. high, and contained a bed of zeolite 2 ft. deep. The capacity of one filter was supposed to be 6,000 gal. per hour. The actual rate of filtration at Glogau was 7.5 gal. per square foot per minute, although at Dresden filters were operated at rates as high as 20 gal. per square foot per minute.

The estimated cost of treatment, based upon a price of 10 cts. per pound for sodium permanganate was from $1 to $2 per million gallons, depending upon the amount of manganese in the water.

The high rates of filtration, absence of any continuous chemical feed, and low cost would appear to make it a valuable process for the removal of both manganese and iron.

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CHAPTER XXXII

THE CONTROL OF WATER-PURIFICATION PROCESSES

The problems involved in water purification are obviously dependent upon a number of the arts and sciences for their successful solution. A knowledge of engineering in several of its phases, of chemistry and of bacteriology, is essential to one who desires to operate a purification plant intelligently, and to obtain from it the best results. No two plants, even of the same general type and purifying the same class of water, can be handled by exactly similar methods. Constant observation and study of the many details of plant operation are always needed, no matter how varied an operator's experience may be. While the sanitary purity and physical attractiveness of the purified water should be the first care of an operator of a plant, nevertheless the economic side should not be forgotten.

Care of Plant and Equipment.—The various apparatus now commonly installed in water-purification plants for assisting the operator in handling the plant have been discussed in more or less detail in preceding chapters. It is desired here only to point out that unless this apparatus is kept in the best working order by constant care and attention, and unless it is properly used, it is worse than useless. Inaccurate gages, alarm bells that do not ring, tell-tales that never move, orifices that never deliver their indicated volumes of solution, controllers that do not control, and safety devices that never prevent an accident, are evidences of gross incompetency, if not of actual criminal negligence.

Pumps, motors, steam boilers, engines, electric generators, waterwheels and blowers need periodic overhauling and renewal of worn-out parts. The electrical machinery and apparatus especially must have the best of care to be of real service. Electrical transmission lines, pipe lines and valves should not be forgotten because they are frequently out of sight underground. Buildings, reservoirs and tanks need more or less repair work done on them from time to time. The care of grounds, ditches and drains should not be forgotten even though they may only indirectly affect the operation of the plant.

Detailed advice as to just how a plant should be kept in the

best condition would require too much space. It is desired only to emphasize the painstaking effort that is needed to maintain the physical equipment of the plant at its highest efficiency; and to point out that the impairment of this efficiency by carelessness and neglect causes the plant to become an actual menace to the community to which it is supposed to be supplying a purified water.

Supplies for the Plant.—It usually is the duty of the manager of a plant to keep it supplied with those materials that are constantly required for its operation. In those plants in which chemical coagulants are used, the uninterrupted supply of chemicals is of the utmost importance. Inadequate storage facilities for chemicals often make the plant dependent upon uniform delivery of these supplies. Where shipments are made from a considerable distance, orders should be made sufficiently in advance to guard against possible delays in transit. It is not always possible to foresee a change in the character of the water being treated that will require a suddenly increased demand for chemicals, and, in consequence, a more rapid depletion of the stock in hand than was anticipated. However, a study of the meteorological data available may show the times when fluctuations in the quality of the raw water are likely to occur; and from this information one may be able to make provision in advance by increasing the stock of chemicals.

The supply of other materials should be as carefully forecasted as possible. Repair parts should always be kept in stock. For all large plants, a stock room is a necessity; and if the plant is isolated, a small machine shop fitted with a lathe, drill press, grinder, pipe-cutting machine, and the necessary work benches, will be found of the greatest value in cases of emergency, and of real assistance in the routine operation of the plant.

Records of Operation.—Unless the principal operation data are consistently recorded, and as constantly watched and checked by those in charge, the results of operation are liable to be irregular and, at times, extremely unsatisfactory. A system of record sheets, which may be filled out without too much trouble by foremen, and which afterward can be given a more permanent form of record in the office of the plant should be worked out for each plant. General forms may assist one, but usually forms must be adapted to each particular plant and to the special conditions involved.

Some of the printed forms which should be provided are for the rate of flow of water through the plant, for the elevation of the water in the various reservoirs, basins and tanks, for the initial and final losses of head on individual filters, for the length of the period of service of the filters, for the depth to which sand is scraped, for the water filtered between cleanings, for the length of time required to wash a filter, for the quantities of chemicals used, for the setting of orifices, for the record of the periodic cleaning of tanks, and for many other operations about which information is desired. How these records shall be grouped, depends upon the particular plant involved, and on its type.

Further elaboration of this data for permanent records should be made and kept in loose-leaf files and ledgers in the office of the plant. From these records the monthly and annual reports may be compiled.

Laboratory data should, of course, be carefully recorded and compiled in the same manner as the usual operation data. The study of both sets of records by themselves, and in relation to each other, is the duty of an intelligent and careful operator. The needless piling up of data is not advisable, although more information will probably be collected during the first year or two of operation than is necessary. The really essential information can be quickly separated from the unessential after a year or two of operation, and the records for future operation modified accordingly.

Laboratory.—No purification plant can be said to be really equipped unless it is provided with a laboratory. This is true of any type of plant. Bacteriological data are absolutely essential for guidance in the intelligent supervision of the plant, and where chemicals are used, a certain amount of chemical data are also required. Physical tests of the appearance of the water, such as for color and turbidity, are generally needed, and sometimes tests for the odor of the water are useful. Chemical tests for total hardness, carbonate and caustic alkalinities, sulphates, chlorides, free chlorine and oxygen consumed are quite commonly required. In plants handling waters containing iron and manganese, these two elements must of course be tested for. In waters containing considerable organic matter, tests for nitrogen in its various forms may throw some light on the action of the plant.

The arrangement and equipment of the laboratory need not be elaborate, although certain details and some essential apparatus

must be installed if the laboratory is to be of real service. Incubators for making the bacterial examinations, and an ice chest for holding samples can not be omitted. A steam bath, hot plate, oven, steam-sterilizing apparatus, glassware for chemical and bacteriological work, and a moderate amount of laboratory chemicals and supplies, constitute the practical minimum of equipment.

Interpretation of Data.—The significance of the strictly engineering data obtained in operating the plant is usually quite apparent. Faulty operation of mechanical devices, such as incorrect gage and meter readings, or stoppage of flow in pipe lines, or leaking tanks, are usually not hard to trace and to correct. A careful scrutiny of this data each day will often enable the operator to check serious trouble in its early stages, and thus prevent any interference in the continuous operation of the plant.

The significance of the chemical data obtained in operating a water-purification plant is generally apparent from a careful study of the chemical characteristics of the water before and after treatment, in conjunction with the particular processes employed. The effect of chemical coagulants, subsidence, filtration and disinfection can generally be directly traced with the aid of a comparatively few chemical determinations. Excess treatment with chemicals must, of course, always be guarded against, and the chemical laboratory furnishes the information by which this treatment of the water may be adjusted.

The removal of color, sediment, and sometimes odor, are readily detected by the standard methods of examination now commonly in use, and the inferences to be drawn from the results obtained are obvious.

The significance of the bacteriological data obtained in the examination of a public water supply is not as clear as could be wished, and the same statement is practically true, even if the supply has been subjected to purification processes. Since the present technique of water bacteriology does not enable the presence or absence of pathogenic organisms to be directly demonstrated by any practicable routine procedure, the results obtained are indicative only, and require information from other sources in order to arrive at any reasonable interpretation of their intrinsic value.

Bacterial counts in themselves afford but little evidence of the purity of the water. A pure water may show a high bacterial count and a polluted water a comparatively low bacterial count.

However, the number of bacteria in an untreated water compared with the number found in the same water after it has been purified gives a fair measure of the degree of improvement effected. This is, of course, based on the assumption that the proportion of pathogenic organisms removed by the process is in the same ratio as are the total number removed. It is apparent that the hygienic character of a water can not be definitely fixed by merely a knowledge of the number of bacteria it may contain, and for this reason a reliable indicator of pollution would be of great value.

Mr. George W. Fuller in his paper on the "Biochemical and Engineering Aspects of Sanitary Water Supply"[1] briefly and clearly states the difficulty of establishing the value of an indicator of pollution, as follows:

"After eliminating as a satisfactory indicator of pollution the total count of bacteria, which may originate anywhere at all, the next natural step is to restrict our investigation to those particular bacteria which seem likely to originate in the human body. The normal rule seems to be that only human diseases are transmitted to other human beings, and that bacteria, in order to cause infection, must originate in man. Infectious pollution in water supplies arises almost wholly from discharges from the human intestines, and any bacteria which can be assumed to originate in the human intestines would be a fair measure of such human pollution which potentially might be dangerous or infectious.

"Unfortunately, there are no such bacteria which are a positive indicator of human pollution. The nearest thing to it are the coli bacilli. Thus B. coli are always found in any discharges from the human intestines. They are also found, however, in the discharges from the intestines of animals, birds, fishes and on cereals, grains and many other places. They are even found in the air, the dust, and, because of their widespread occurrence, have often been termed 'ubiquitous.' Because of these conditions it is not a simple matter to say that any water supply which shows the presence of B. coli necessarily possesses any specific danger of human pollution or of infection which might at any time have been present. It merely suggests a potential danger in such a water."

It is, of course, true that the same reasoning with reference to the significance of the presence of B. coli holds good as in the case of the total number of bacteria, namely, that so far as removal of B. coli is effected, it is probably in the same ratio as are the pathogenic forms. Where disinfection is practised there may

[1] *Journal of the Franklin Institute*, July, 1915.

be some selective action on the bacteria; but if this does occur, it would seem probable that disease-producing organisms are likely to be fully as susceptible to the toxic effects of the disinfectant, if not more so, as are the ordinary forms of water bacteria.

The purity of a water supply is, in the final analysis, determined by its effect upon the people who drink the water. Absence of known water-borne diseases, such as typhoid fever, dysentery and other intestinal disturbances in a community are good evidence of the quality of the water supply. However, since such diseases are known to be transmitted through other avenues, such as milk, fruit, fresh vegetables, shell fish, and so forth, the prevalence of these diseases in a community does not necessarily point to an infected water supply.

Dr. W. H. Frost, Passed Assistant Surgeon of the United States Public Health Service, in a paper on "Some Considerations in Estimating the Sanitary Quality of Water Supplies,"[1] has set forth the relative value of the evidence obtained in the laboratory and by sanitary surveys, as follows:

"Of the laboratory examinations applicable to determining the nature and extent of pollution of a water supply, bacteriological examinations have the most direct bearing upon sanitary quality, which is a question of bacterial pollution. The most specific of the bacteriological examinations in general use are quantitative tests for bacteria of the B. coli group, since these tests afford a direct measure of the numbers of intestinal bacteria present, and since typhoid bacilli are found only in association with intestinal discharges. Nevertheless, such tests, however accurate and specific they may be, show only the extent of pollution with intestinal discharges in general; they do not distinguish between pollution with intestinal discharges from lower animals which are not subject to infection with typhoid bacilli, and the much more dangerous pollution from human sources. They still further fail to distinguish between human discharges actually containing typhoid bacilli and discharges free from this specific infection."

Speaking further of the correlation of a sanitary survey and bacteriological examinations, Dr. Frost states, that:

"Sanitary surveys and bacteriological examinations give only indirect and inferential knowledge as to the probable presence and numbers of typhoid bacilli in a water supply. Even were this knowledge much more direct and exact, it would still fall short of being all that is needed for an estimate of the sanitary quality of the water. The second

[1] *Jour. Amer. Water-works Assn.*, December, 1915.

requisite is an equally exact knowledge of the effects which the known pollution will produce."

He concisely sums up the values that we may legitimately attach to the available evidence on the quality of a water supply as follows:

"To recapitulate: bacteriological examinations, combined with a careful survey of the sources of pollution and of the safeguards against them, give accurate knowledge of the extent of pollution of a water supply in terms of intestinal bacteria from all sources. They give only indirect and inferential knowledge of specifically dangerous pollution with typhoid bacilli. To make the knowledge acquired by bacteriological examinations and sanitary surveys significant with respect to the sanitary quality of water, it is necessary to know the *effects* produced by a given amount of pollution. Such effects can be definitely ascertained and measured in the case of epidemic outbreaks, proven to be due to the use of polluted water supplies. The effects can be definitely recognized but not accurately measured in the case of highly polluted water supplies causing high rates of endemic typhoid prevalence. In the case of slightly polluted water supplies, it is not philosophically possible to probe by present methods whether or not they may cause a relatively small incidence of typhoid fever, so small as to be obscured by other more prominent factors.

"This paper is not intended to present a pessimistic point of view or to deny the possibility of forming a reasonably accurate estimate of the sanitary quality of a water supply. On the contrary, the writer wishes to express his entire confidence in the reliability of an expert opinion, formed after careful study from all angles, and stated conservatively, with an understanding of the limitations of the evidence. Unless these limitations are remembered, however, there is always the danger of drawing too sweeping conclusions from evidence bearing solely upon the extent of pollution of a water supply.

"As regards establishing a firmer basis for opinions in the future, this must obviously be accomplished by improving and extending not only laboratory studies of the quality of water supplies, but equally epidemiologic studies of their relation to typhoid prevalence. Methods of epidemiologic study, like bacteriological and chemical methods, must be so standardized that results can be summarized into a general fund of knowledge. Routine bacteriological examinations of water supplies, closely coördinated with careful, long-continued epidemiologic studies may be confidently expected to increase, perhaps to an extent not now foreseen, our ultimate knowledge of the sanitary safety of water supplies."

The Human Factor.—An enterprise to be successful must be directed and operated with intelligence and skill. Unless the manager of a purification plant is properly qualified for his work, and unless he is given competent assistance to operate the plant, it may become an actual menace to the community. Dr. Allen J. McLaughlin[1] well states the situation, when he says:

"With a polluted source the mere installation of a purification plant does not guarantee safe water. Even if perfect in design and construction, unless efficiently operated and controlled, a safe effluent need not be expected. The writer has seen filter plants designed by our best engineers—perfect mechanisms, which if properly operated, would produce safe water—placed in the hands of an assassin who was a promoted stoker, and absolutely ignorant of bacteriology or chemistry."

The responsibility for mal-administration of water-purification plants is frequently found in an uneducated public, which, if the plant is privately owned, is too indolent to enforce its contract rights, or if publicly owned is too careless of its own obligations to see that the plant is placed in the hands of technically trained men, who are competent to produce the proper results. Fortunately, there are communities fully awake to their responsibilities in conserving the public health by a safe water supply, and in such the operator of a plant finds his efforts to maintain an efficient force well supported.

The manager of a plant who is able to instill into his assistants the need for that careful attention to details, without which no plant can be properly kept up, will be the most successful in maintaining an *esprit de corps*, which will be reflected in the general appearance of the plant, and in the quality of its output. Economy of operation, consistent with maintaining a high standard of purity for the effluent of the plant, must be carefully and continuously studied. The safety of the supply, however, must always be kept in mind, for "eternal vigilance is the price of pure water."

[1] "Sewage Pollution of Boundary Waters." *Jour. Am. Water-works Assn.*, May, 1914.

APPENDIX A

ON THE FLOW OF WATER THROUGH RAPID SAND FILTERS

BY

C. N. MILLER, ASSOC. M. AM. SOC. C.E.

In this discussion it is assumed that the walls of the filter tank and all channels and piping are tight, so that no air can enter, and that the lower end of the outlet pipe is submerged or trapped so that no air can enter also at this point.

The notation used is as follows:

Let h_a = the pressure of the atmosphere.
E_s = the elevation, in feet above some assumed datum, of the surface of the water in the filter tank.
E_0 = the elevation of the surface of the water in the effluent chamber.
E_n = the elevation of any intermediate horizontal plane.
v_s = the velocity of the water at the surface in the filter, equals zero.
v_0 = the velocity of the water in the outlet pipe.
v_n = the velocity of the water at any intermediate horizontal plane.
p_s = the absolute pressure at the surface of the water in the filter.
p_0 = the absolute pressure at the surface of the water in the effluent chamber.
p_n = the absolute pressure at any intermediate horizontal plane.
h_f = any one of the frictional resistances overcome in the passage of the water through the filter.

All pressures are to be measured in feet of water. Absolute pressures are to be measured above zero pressure.

The flow of the water being uniform, we have by Bernoulli's theorem:

$$E_s + p_s + \frac{v_s^2}{2g} = E_n + p_n + \frac{v_n^2}{2g} + S_n^s h_f$$

where

$S_n^s h_f$ = the sum of the frictional resistances between the surface of the water in the filter and the plane n.

But $v_s = 0$, and $p_s = h_a$, therefore, we have

$$E_s + h_a = E_n + p_n + \frac{v_n^2}{2g} + S_n^s h_f$$

or

$$h_a - p_n = \left(\frac{v_n^2}{2g} + S_n^s h_f\right) - (E_s - E_n) \qquad (1)$$

Also

$$E_n + p_n + v\frac{v_n^2}{2g} = E_0 + p_0 + \frac{v_0^2}{2g} + S_0^n h_f$$

but $p_0 = h_a$, therefore, we have

$$E_n + p_n + \frac{v_n^2}{2g} = E_0 + h_a + \frac{v_0^2}{2g} + S_0^n h_f$$

or

$$h_a - p_n = (E_n - E_0) - \left(\frac{v_0}{2g} - \frac{v_n^2}{2g} + S^n h_f\right) \tag{2}$$

We also have

$$E_s + p_s + \frac{v_s^2}{2g} = E_0 + p_0 + \frac{v_0^2}{2g} + S_0 h_f$$

but $v_s = 0$, and $p_s = p_0 = h_a$, therefore,

$$E_s - E_0 = \frac{v_0^2}{2g} + S_0^s h_f \tag{3}$$

The velocity heads are quite small as compared with the frictional resistances, and for practical purposes may be neglected, in which case the above equations may be written:

$$h_a - p_n = S_n^s h_f - (E_s - E_n) \tag{4}$$

$$h_a - p_n = (E_n - E_0) - S_0^n h_f \tag{5}$$

$$E_s - E_0 = S_0^s h_f \tag{6}$$

From equation (4) it is seen that when the frictional resistance encountered down to any intermediate plane is less than $(E_s - E_n)$, that is the static head of water on the plane then the term $h_a - p_n$ is negative or the absolute pressure at the plane is greater than atmospheric pressure; when the frictional resistance is equal to the static head the term $h_a - p_n = 0$, or the absolute pressure is equal to the atmospheric pressure; and when the frictional resistance is greater than the static head, the term $h_a - p_n$ is positive or the absolute pressure is less than atmospheric pressure.

From equation (5) it is seen that when the frictional resistance below any intermediate plane is less than $E_n - E_0$, that is the portion of the static head below the plane, the term $h_a - p_n$ is positive, or the absolute pressure is less than atmospheric; when the frictional resistance is equal to $E_n - E_0$, the term $h_a - p_n = 0$, or the absolute pressure is equal to the atmospheric; and when the frictional resistance is greater than $E_n - E_0$, the term $h_a - p_n$ is negative, or the absolute pressure is greater than atmospheric.

Equations (4) and (5) also show that, when the frictional resistance above any intermediate plane is less than the static head on the plane, the frictional resistance below the plane must be greater than the remaining portion of the total static head by an equal amount. The converse of the above statement is also true. That this must be the case appears from equation (6), which shows that the total available head is equal to the total frictional resistance.

When the absolute pressure at any intermediate plane is less than atmospheric pressure, a condition exists which is commonly described by stating that the filter is operating under a "negative head." From the above discussion it is seen that this condition is only a particular case of the general problem of the flow of water through a filter.

The accompanying sketch (Fig. 149) may illustrate some of the points brought out in the foregoing discussion. The arrangement of the filter, effluent chamber and appurtenances shown, is that of one of the Harrisburg, Pa., filters, according to information taken from an article in the *Engineering Record* of Mar. 9, 1907. The arrangement of the trapped tubes for indicating the losses of head at the various planes passing through the points at which the tubes penetrate the wall of the filter is adapted from the discussion by Mr. Fuertes, of his paper on "Water Purification at Steelton, Pa.," appearing in the *Transactions* of the American Society of Civil Engineers for March, 1910. The locations of the points at which the tubes pass

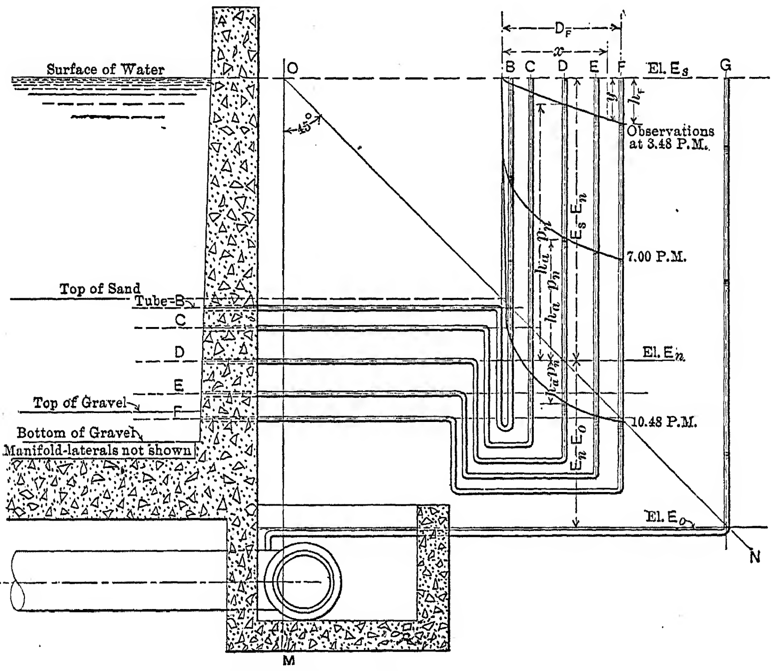

FIG. 149.—Diagrammatic sketch to illustrate flow of water through a filter.

through the wall of the filter and the distances which the water in the tubes stand below the level of the water in the filter, as well as the relative locations of the top of gravel, top of sand, etc., are scaled from Mr. Fuertes' diagram for the run of June 7, 1909. The arrangement of the tubes showing the trapping is diagrammatic. The level of the water in the filter varied slightly during the period in which the observations were taken, but for simplicity has been taken at an approximate average value as shown on the sketch.

The horizontal distances of the vertical axes of the tubes *B*, *C*, *D*, *E* and *F* from the line *OM* are respectively equal to the distances of the points where the tubes pierce the side of the filter below the surface of the water in

the same. The distance OG is equal to the distance between the level of the water in the filter and that in the effluent chamber. The line ON drawn at 45° with the horizontal evidently cuts the vertical axis of each tube at a point on a level with that where the tube pierces the side of the filter, except the axis of tube G which is cut at the level of the surface of the water in the effluent chamber.

Consider tube D and let the horizontal plane passing through the point where the tube pierces the side of the filter be taken as plane n. The notations made on the diagram in regard to tube D are self-evident. The total head $E_s - E_0$ is divided by the water level in the tube into two parts, the upper one being equal to the frictional resistance above the plane n or to $S_n^s h_f$, and the lower one to the frictional resistance below plane n or to $S_0^n h_f$. We then have

$$h_a - p_n = S_n^s h_f - (E_s - E_n) \text{ or equation (4)}$$

and

$$h_a - p_n = (E_n - E_0) - S_0^n h_f \text{ or equation (5)}$$

The curves shown in the sketch are those given by the equation

$$y = h_F \left(\frac{x}{D_F}\right)^{\frac{\tan \alpha D_F}{h_F}}$$

where D_F is equal to the distance from the top of the sand bed to the plane n passing through the point where the tube F pierces the side of the filter, h_F is equal to the distance from the surface of the water in the filter to the level of the water in the tube F, that is to the head or force required to maintain the flow from above down to the plane n, and tan α is equal to the tangent of the angle the curve makes with the horizontal when $x = D_F$, that is to $\frac{dy}{dx}$. Tangent α is evidently equal to the head required to force the water through a unit thickness of material such as that at the level of the plane n, and its value is taken as 0.34 for each of the three curves, it being assumed that the frictional resistance does not vary for a layer at this level during the run, which is practically the case. Each curve corresponds to a set of observations taken simultaneously in all the tubes.

By reference to the diagram it will be evident that as long as the curve connecting the water levels in the tubes remains above the line ON, the pressures at the various planes in the filter will be greater than atmospheric. When the curve cuts the line ON it indicates that a zone will exist in the filter in which pressures will be less than atmospheric.

Since preparing the above diagram similar curves have been published by H. Malcolm Pirnie in the discussion of a paper by Joseph W. Ellms and John S. Gettrust, which appeared in vol. 80, page 1342 (1916) of the *Transactions* of the American Society of Civil Engineers.

APPENDIX B

AN APPROXIMATE FORMULA FOR CALCULATING THE DISCHARGING CAPACITY OF RAPID SAND FILTER WASH WATER TROUGHS

BY

C. N. MILLER, ASSOC. M. AM. SOC. C.E.

Let $ABCD$ represent a trough of rectangular cross-section receiving water uniformly along the level edges BC.

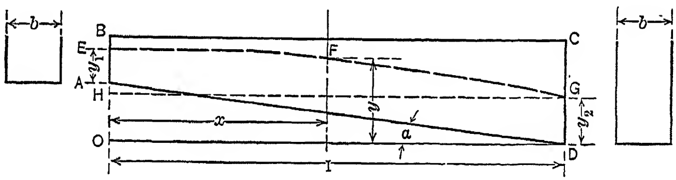

FIG. 150.—Diagram to illustrate flow of water in a wash-water trough.

Let Q_1 = the total quantity of water received by the trough in second-feet.
Q = the quantity passing any section as at F in second-feet.
v = the horizontal velocity at section F.
v_2 = the horizontal velocity at the lower end of the trough.
α = the angle the bottom of the trough makes with the horizontal.
O = the origin of coördinates.
EFG = the surface curve of the water
l = the length of the trough.
b = the width of the trough.
y_1 = the depth of water at the upper end of trough.
y_2 = the depth of water at the lower end of the trough.
w = the weight of a cubic foot of water.
x and y = the coördinates of the surface curve at section F.

The quantity of water Q falling through the height dy generates an increment of kinetic energy

$$wd\left(\frac{Qv^2}{2g}\right)$$

and overcomes the frictional resistance in a length dx, noting that Q and v both vary. Assuming that the work done in overcoming the frictional resistance may be expressed by

$$w\zeta d\left(\frac{Qv^2}{2g}\right),$$

we have the equation

$$-wQdy = wd\left(\frac{Qv^2}{2g}\right) + w\zeta d\left(\frac{Qv^2}{2g}\right) \text{ or, dividing by } w,$$

$$-Qdy = (1+\zeta)d\left(\frac{Qv^2}{2g}\right) \tag{1}$$

But $Q = Q_1 \frac{x}{l}$, therefore,

$$-Q_1 \frac{x}{l} dy = (1+\zeta)d\left(\frac{Qv^2}{2g}\right) \tag{2}$$

Integrating equation (2) throughout the length of the trough, we have,

$$-\int_{y_1 + l \tan \alpha}^{y_2} Q_1 \frac{x}{l} dy = (1+\zeta)\frac{Q_1 v_2^2}{2g} \tag{3}$$

But $-\int_{y_1 + l \tan \alpha}^{y_2} xdy$ = the area of the figure $EFGH$, which may be expressed by $m(y_1 + l \tan \alpha - y_2)l$, where m equals a coefficient whose value would be ⅔ if the surface curve were a parabola. We have then by making this substitution

$$m \frac{Q_1}{l}(y_1 + l \tan \alpha - y_2)l = (1+\zeta)\frac{Q_1 v_2^2}{2g},$$

or

$$y_1 + l \tan \alpha - y_2 = \left(\frac{1+\zeta}{m}\right)\frac{v_2^2}{2g} \tag{4}$$

But $v_2 = \frac{Q_1}{by_2}$, whence

$$y_1 + l \tan \alpha - y_2 = \frac{1+\zeta}{m}\left(\frac{Q_1}{by_2}\right)^2 \frac{1}{2g} \tag{5}$$

From equation (5) it can be seen that Q_1 equals 0 when y_2 equals $y_1 + l \tan \alpha$, that is when there is no fall, and also when y_2 equals 0. If y_1 is considered constant, there will be some value of y_2 between those above mentioned, which will render Q_1 a maximum. Assume then that the levels will so adjust themselves that Q_1 will be a maximum. By differentiating equation (5) with respect to y_2, considering y_1 constant, we have

$$-1 = \frac{1+\zeta}{2mg} 2\left(\frac{Q_1}{by_2}\right)\frac{1}{b}\left(\frac{\frac{dQ_1}{dy_2}y_2 - Q_1}{y_2^2}\right) \tag{6}$$

In equation (6) put $\frac{dQ_1}{dy_2} = 0$, and we have

$$1 = \frac{1+\zeta}{mg}\frac{Q_1^2}{b^2 y_2^3}$$

whence

$$Q_1 = \sqrt{\frac{mg}{1+\zeta}}\, by_2^{3/2} \tag{7}$$

Eliminate Q_1 between equations (5) and (7), and we have

$$y_2 = \tfrac{2}{3}(y_1 + l \tan \alpha) \tag{8}$$

From equations (7) and (8), we have

$$Q_1 = \sqrt{\frac{8}{27}\frac{mg}{1+\zeta}}\, b(y_1 + l \tan \alpha)^{3/2} \qquad (9)$$

By applying equation (9) to some experimental data obtained by Mr. J. W. Ellms at the Cincinnati filtration plant, a value of ζ of 0.75 was obtained by assuming a value for m of $\frac{2}{3}$. Substituting these values of m and ζ in equation (9) we have finally

$$Q_1 = 1.91b(y_1 + l \tan \alpha)^{3/2} \qquad (10)$$

In the case of a trough whose cross-section is not rectangular, application of the above formulæ may be made by assuming a rectangular section of equivalent area to the section of the trough under consideration.

The value of y_2 as given by equation (8) applies to the case where the discharge is into a chamber in which the water level is at or near the level of the water in the lower end of the trough. In the case of a free fall from the end of the trough, owing to weir action, the actual value of y_2 will be less than that given by the equation.

TABLE 1.—APPROXIMATE EQUIVALENTS OF VARIOUS MEASURES OF RATE OF FILTRATION

	Million U. S. gallons per acre per 24 hr.	Million British gallons per acre per 24 hr.	U. S. gallons per square foot per hour	British gallons per square foot per hour
1,000,000 U. S. gal. per acre per 24 hr.	1.000	0.833	0.96	0 80
1,000,000 British gal. per acre per 24 hr.	1.200	1.000	1.15	0 96
1 U. S. gal. per square foot per hour	1.045	0.870	1.00	0 83
1 British gal. per square foot per hour	1.255	1.045	1 20	1 00
1 cu. ft. per square yard per hour	.0.869	0.724	0 83	0 69
1 linear in., vertical velocity, per hour	0.652	0.543	0.62	0.52
100 lineal mm., vertical velocity, per hour	2.566	2.139	2.46	2.05
1 lineal meter, vertical velocity, per 24 hr. = 1 cu. meter per square meter per 24 hr	1.069	0 891	1 02	0.85

	Cubic feet per square yard per hour	Vertical velocity in inches per hour	Vertical velocity in millimeters per hour	Vertical velocity in meters per 24 hr. Or cubic meters per square meter per 24 hr.
1,000,000 U. S. gal per acre per 24 hr.	1.15	1.53	39.0	0.935
1,000,000 British gal. per acre per 24 hr	1.38	1.84	46.8	1.122
1 U. S. gal. per square foot per hour	1 20	1.60	40.7	0.978
1 British gal. per square foot per hour	1.44	1.92	48.9	1.174
1 cu. ft. per square yard per hour	1.00	1.33	33.9	0.813
1 linear in., vertical velocity, per hour	0.75	1.00	25.4	0.610
100 lineal mm., vertical velocity, per hour	2.95	3.94	100.0	2.400
1 lineal meter, vertical velocity, per 24 hr. = 1 cu meter per square meter per 24 hr	1.23	1.64	41.7	1.000

From Report on Water Purification at Louisville.

TABLE 2.—CONVERSION OF STATEMENTS OF CHEMICAL COMPOSITION

	Grains per U S gallon (231 cu in)	Grains per British gallon (277 cu in)	Parts per hundred thousand	Parts per million
1 grain per U. S. gallon	1.000	1 20	1 71	17.1
1 grain per British gallon.	0 830	1.00	1.43	14.3
1 part per 100,000. .	0.580	0.70	1.00	10.0
1 part per 1,000,000.	0.056	0.07	0.10	1 0

TABLE 3.—CONVERSION TABLE OF HARDNESS FROM ENGLISH, FRENCH AND GERMAN DEGREES TO PARTS PER MILLION

	Parts per million	Clark degrees	French degrees	German degrees
Parts per million.	1 0	0 07	0 10	0.56
Clark or English degrees	14.3	1 00	1 43	0.80
French degrees	10.0	0 70	1.00	0.56
German degrees.	17.8	1 24	1.78	1.00

NOTE.—English degrees of hardness, Clark scale, are equivalent to grains of calcium carbonate per imperial gallon, and are multiplied by 14.3 to give parts per million. French degrees of hardness represent parts per hundred thousand of calcium carbonate, and are multiplied by 10 to give parts per million. German degrees of hardness represent parts per hundred thousand of calcium oxide, and are multiplied by 17.8 to give parts per million of calcium carbonate.

Table 4.—Weights and Measures

(From Slocum's "Elements of Hydraulics")

Metric system	U. S. standard	Metric system	U. S. standard
Length	Length	Weight	Weight
1 millimeter = 0.0394 inches	1 inch = 2.5309 centimeters	1 gram = 15.4323 grains	1 lb. = 0.4536 kilos
1 centimeter = 0.3937 inches	1 foot = 30.4794 centimeters	1 kilogram = 2.2046 lb.	1 cwt. = 50.8024 kilos
1 meter = 39.3708 inches	1 yard = 0.9143 meters	1 tonneau = 2204.55 lb.	1 ton = 1016.0483 kilos
1 kilometer = 0.6214 miles	1 mile = 1.6093 kilometers		
Area	Area	Dry measure	Dry measure
1 sq. centimeter = 0.1549 sq. in.	1 sq. in. = 6.4513 sq. centimeters	1 centiliter = 0.0181 pints	1 pint = 55.0661 centiliters
1 sq. meter = 10.7631 sq. ft.	1 sq. ft. = 0.0929 sq. meters	1 liter = 0.908 quarts	1 quart = 1.1013 liters
1 are = 119.5894 sq. yd.	1 sq. yd. = 0.8361 sq. meters	1 hectoliter = 2.837 bushels	1 bushel = 35.2416 liters
1 hectare = 2.4711 acres	1 acre = 0.4047 hectares		
Volume	Volume	Liquid measure	Liquid measure
1 cubic meter = 35.3166 cu. ft.	1 cubic foot = 0.02831 cubic meters	1 centiliter = 0.0211 pints	1 pint = 47.3171 centiliters
		1 liter = 1.0567 quarts	1 quart = 0.9563 liters
		1 hectoliter = 26.4176 gallons	1 gallon = 3.7854 liters

1 U. S. ton of shipping = 40 cubic feet = 32.143 U. S. bushels = 1.1326 cubic meters

Equivalents of Various Weights and Measures

	U. S. gallon	Imperial gallon	Cubic inch	Cubic foot	Pound	Cwt.	Ton	Liter	Cubic meter
U. S. gallon	1.0	0.83	231.0	0.133	8.33	0.07455	0.00372	3.8	0.00378
Imperial gallon	1.2	1.0	277.274	0.16	10.0	0.0892	0.00446	4.537	0.00454
Cubic inch	0.004329	0.003607	1.0	0.00053	0.03607	....	....	0.0163	....
Cubic foot	7.48	6.23	1728.0	1.0	62.35	0.557	0.028	28.375	0.0283
Pound	0.083	0.10	27.72	0.016	1.0	112.0	2240.0	....	....
Cwt.	13.44	11.2	....	1.8	....	1.0	20.0	....	....
Ton	268.8	224.0	....	35.9	....	0.05	1.0	1000.0	1 0
Liter	0.264	0.22	61.0	0.0353	....	....	0.001	1.0	0 001
Cubic meter	264 0	220.0	61028.0	35.31	. .	....	1.0	1000.0	1 0

TABLE 5.—HEAD AND PRESSURE EQUIVALENTS

(From Slocum's "Elements of Hydraulics")

Head of Water in Feet and Equivalent Pressure in Pounds per Square Inch

Feet head	Pounds per sq in.	Feet head	Pounds per sq. in.	Feet head	Pounds per sq. in.
1	0 43	55	23 82	190	82.29
2	0.87	60	25.99	200	86.62
3	1.30	65	28.15	225	97.45
4	1.73	70	30.32	250	108.27
5	2.17	75	32.48	275	119.10
6	2.60	80	34.65	300	129.93
7	3.03	85	36.81	325	140.75
8	3.40	90	38.98	350	151.58
9	3.90	95	41.14	375	162.41
10	4.33	100	43.31	400	173.24
15	6.50	110	47.64	500	216.55
20	8.66	120	51.97	600	259.85
25	10.83	130	56.30	700	303.16
30	12.99	140	60.63	800	346.47
35	15.16	150	64.96	900	389.78
40	17.32	160	69.29	1,000	433.09
45	19.49	170	73.63		
50	21.65	180	77.96		

Pressure in Pounds per Square Inch and Equivalent Head of Water in Feet

Pounds per sq. in.	Feet head	Pounds per sq. in.	Feet head	Pounds per sq. in.	Feet head
1	2.31	55	126.99	180	415.61
2	4.62	60	138.54	190	438.90
3	6.93	65	150.08	200	461.78
4	9.24	70	161.63	225	519.51
5	11.54	75	173.17	250	577.24
6	13.85	80	184.72	275	643.03
7	16.16	85	196 26	300	692.69
8	18.47	90	207.81	325	750.41
9	20.78	95	219.35	350	808.13
10	23.09	100	230.90	375	865.89
15	34.63	110	253.98	400	922.58
20	46.18	120	277.07	500	1,154.48
25	57.72	125	288.62		
30	69.27	130	300.16		
35	80.81	140	323.25		
40	92.36	150	346.34		
45	103.90	160	369.43		
50	115.45	170	392.52		

Table 6.—Discharge Equivalents
(From Slocum's "Elements of Hydraulics")

Gallons per min.	Cubic feet per sec.	Cubic feet per min.	Gallons per hour	Gallons per 24 hours	Bbls. per minute, 42 gal. bbl	Bbls. per hour, 42 gal. bbl.	Bbls. per 24 hours, 42 gal. bbl.
10		1.3368	600	14,400	0.24	14.28	342.8
12	.	1.6042	720	17,280	0.29	17.14	411.4
15	.... .	2.0052	900	21,600	0.36	21.43	514.3
18	.	2.4063	1,080	25,920	0.43	25.71	617.1
20	.....	2.6733	1,200	28,800	0.48	28.57	685.7
25	....	3.342	1,500	36,000	0.59	35.71	857.0
27	.....	3.609	1,620	38,880	0.64	38.57	925.0
30	.....	4.001	1,800	43,200	0.71	42.85	1,028.0
35	. ...	4.678	2,100	50,400	0.83	50.0	1,200.0
36	.....	4.812	2,160	51,840	0.86	51.43	1,234.0
40	....	5.348	2,400	57,600	0.95	57.14	1,371.0
45	0.1	6.015	2,700	64,800	1.07	64.28	1,543.0
50	....	6.684	3,000	72,000	1.19	71.43	1,714.0
60	.. ..	8.021	3,600	86,400	1.43	85.71	2,057 0
70	....	9.357	4,200	100,800	1.66	100.0	2,400.0
75	. ..	10.026	4,500	108,000	1.78	107.14	2,570.0
80	....	10.694	4,800	115,200	1.90	114.28	2,742.0
90	0.2	12.031	5,400	129,600	2.14	128.5	3,085.0
100	.....	13.368	6,000	144,000	2.30	142.8	3,428.0
125	.....	16.710	7,500	180,000	2 98	178.6	4,286.0
135	0.3	18.046	8,100	194,400	3.21	192.8	4,628.0
150	....	20.052	9,000	216,000	3.57	214.3	5,143.0
175	.....	23.394	10,500	252,000	4.16	250.0	6,000.0
180	0.4	24.062	10,800	259,200	4.28	257.0	6,171.0
200	. ...	26.736	12,000	288,000	4.76	285.7	6,857.0
225	0.5	30.079	13,500	324,000	5.35	321.4	7,714.0
250	.....	33.421	15,000	360,000	5.95	357.1	8,570.0
270	0.6	36.093	16,200	388,800	6.43	385.7	9,257.0
300	.....	40.104	18,000	432,000	7.14	428.5	10,284.0
315	. ...	42.109	18,900	453,600	7.5	450.0	10,800.0
360	0.8	48.125	21,600	518,400	8.57	514.3	12,342.0
400	.....	53.472	24,000	576,000	9.52	571.8	13,723.0
450	1.0	60.158	27,000	648,000	10.7	642.8	15,428.0
500	.....	66.842	30,000	720,000	11.9	714.3	17,143.0
540	1.2	72.186	32,400	777,600	12.8	771.3	18,512.0
600	.....	80.208	36,000	864,000	14.3	857.1	20,570.0
630	1.4	84.218	37,800	907,200	15.0	900.0	21,600.0
675	1.5	90.234	40,500	972,000	16.0	964.0	23,143.0
720	1.6	96.25	43,200	1,036,800	17.0	1,028.0	24,685.0
800	.....	106.94	48,000	1,152,000	19.05	1,142.0	27,387.0
900	2.0	120.31	54,000	1,296,000	21.43	1,285.0	30,857.0
1,000	.....	133.68	60,000	1,440,000	23.8	1,428.0	34,284.0
1,125	2.5	150.39	67,500	1,620,000	26.78	1,607.0	38,571.0
1,200	...	160.42	72,000	1,728,000	28.57	1,714.0	41,143.0
1,350	3.0	180.46	81,000	1,944,000	32.14	1,928.0	46,085.0
1,500	.....	200.52	90,000	2,160,000	35.71	2,142.0	51,427.0
1,575	3.5	210.54	94,500	2,268,000	37.5	2,250.0	54,000.0
1,800	4.0	240.62	108,000	2,592,000	42.85	2,571.0	61,710.0
2,000	.....	267.36	120,000	2,880,000	47.64	2,857.0	68,568.0
2,025	4.5	270.70	121,500	2,916,000	48.21	2,892.0	69,425.0
2,250	.....	300.78	135,000	3,240,000	53.57	3,214.0	77,143.0
2,500	.....	334.21	150,000	3,600,000	59.52	3,571.0	85,704.0
2,700	6.0	360.93	162,000	3,880,000	64.3	3,857.0	92,572.0
3,000	.....	401.04	180,000	4,320,000	71.43	4,285.0	102,840.0

TABLE 7.[1]—EFFLUX COEFFICIENTS FOR CIRCULAR ORIFICE

Values of efflux coefficient K in Eq. (32), Par. 55, $Q = \frac{2}{3}Kb\sqrt{2g}\,(H^{3/2} - h^{3/2})$, for circular, vertical orifices, with sharp edges, full contraction and free discharge in air. For heads over 100 ft., use $K = 0.592$

Head on center of orifice in feet	Diameter of orifice in feet												
	0.02	0.03	0.04	0.05	0.07	0.10	0.12	0.15	0.20	0.40	0.60	0.80	1.0
0.3	.....	....	.....	0.637	0.628	0.621	0.613	0.608					
0.4	.....	.....	0.637	0.631	0.624	0.618	0.612	0.606					
0.5	.....	0.643	0.633	0.627	0.621	0.615	0.610	0.605	0.600	0.596	0.592		
0.6	0.655	0.640	0.630	0.624	0.618	0.613	0.609	0.605	0.601	0.596	0.593	0.590	
0.7	0.651	0.637	0.628	0.622	0.616	0.611	0.607	0.604	0.601	0.597	0.594	0.591	0.590
0.8	0.648	0.634	0.626	0.620	0.615	0.610	0.606	0.603	0.601	0.597	0.594	0.592	0.591
0.9	0.646	0.632	0.624	0.618	0.613	0.609	0.605	0.603	0.601	0.598	0.595	0.593	0.591
1.0	0.644	0.631	0.623	0.617	0.612	0.608	0.605	0.603	0.600	0.598	0.595	0.593	0.591
1.2	0.641	0.628	0.620	0.615	0.610	0.606	0.604	0.602	0.600	0.598	0.596	0.594	0.592
1.4	0.638	0.625	0.618	0.613	0.609	0.605	0.603	0.601	0.600	0.599	0.596	0.594	0.593
1.6	0.636	0.624	0.617	0.612	0.608	0.605	0.602	0.601	0.600	0.599	0.597	0.595	0.594
1.8	0.634	0.622	0.615	0.611	0.607	0.604	0.602	0.601	0.599	0.599	0.597	0.595	0.595
2.0	0.632	0.621	0.614	0.610	0.607	0.604	0.601	0.600	0.599	0.599	0.597	0.596	0.595
2.5	0.629	0.619	0.612	0.608	0.605	0.603	0.601	0.600	0.599	0.599	0.598	0.597	0.596
3.0	0.627	0.617	0.611	0.606	0.604	0.603	0.601	0.600	0.599	0.599	0.598	0.597	0.597
3.5	0.625	0.616	0.610	0.606	0.604	0.602	0.601	0.600	0.599	0.599	0.598	0.597	0.596
4.0	0.623	0.614	0.609	0.605	0.603	0.602	0.600	0.599	0.599	0.598	0.597	0.597	0.596
5.0	0.621	0.613	0.608	0.605	0.603	0.601	0.599	0.599	0.598	0.598	0.597	0.596	0.596
6.0	0.618	0.611	0.607	0.604	0.602	0.600	0.599	0.599	0.598	0.598	0.597	0.596	0.596
7.0	0.616	0.609	0.606	0.603	0.601	0.600	0.599	0.599	0.598	0.958	0.597	0.596	0.596
8.0	0.614	0.608	0.605	0.603	0.601	0.600	0.599	0.598	0.598	0.597	0.596	0.596	0.596
9.0	0.613	0.607	0.604	0.602	0.600	0.599	0.599	0.598	0.597	0.597	0.596	0.596	0.595
10.0	0.611	0.606	0.603	0.601	0.599	0.598	0.598	0.597	0.597	0.597	0.596	0.596	0.595
20.0	0.601	0.600	0.599	0.598	0.597	0.596	0.596	0.596	0.596	0.596	0.596	0.595	0.594
50 0	0.596	0.596	0.595	0.595	0.594	0.594	0.594	0.594	0.594	0.594	0 594	0.593	0.593
100.0	0.593	0.593	0.592	0.592	0 592	0.592	0.592	0.592	0.592	0.592	0.592	0.592	0.592

[1] From HAMILTON SMITH'S "Hydraulics."

Table 8.[1]—Efflux Coefficients for Square Orifice

Values of efflux coefficient K in Eq. (32), Par. 55, $Q = \frac{2}{3}Kb\sqrt{2g}(H^{3/2} - h^{3/2})$, for square, vertical orifices, with sharp edges, full contraction, and free discharge in air. For heads over 100 ft. use $K = 0.598$

Head on center of orifice in feet	Side of square in feet												
	0.02	0.03	0.04	0.05	0.07	0.10	0.12	0.15	0.20	0.40	0.60	0.80	1.0
0.3	...	....	.....	0.642	0.632	0.624	0.617	0.612					
0.4	.....	.....	0.643	0.637	0.628	0.621	0.616	0.611					
0.5	.....	0.648	0.639	0.633	0.625	0.619	0.614	0.610	0.605	0.601	0.597		
0.6	0.660	0.645	0.636	0.630	0.623	0.617	0.613	0.610	0.605	0.601	0.598	0.596	
0.7	0.656	0.642	0.633	0.628	0.621	0.616	0.612	0.609	0.605	0.602	0.599	0.598	0.596
0.8	0.652	0.639	0.631	0.625	0.620	0.615	0.611	0.608	0.605	0.602	0.600	0.598	0.597
0.9	0.650	0.637	0.629	0.623	0.619	0.614	0.610	0.608	0.605	0.603	0.601	0.599	0.598
1.0	0.648	0.636	0.628	0.622	0.618	0.613	0.610	0.608	0.605	0.603	0.601	0.600	0.599
1.2	0.644	0.632	0.625	0.620	0.616	0.611	0.609	0.607	0.605	0.604	0.602	0.601	0.600
1.4	0.642	0.630	0.623	0.618	0.614	0.610	0.608	0.606	0.605	0.604	0.602	0.601	0.601
1.6	0.640	0.628	0.621	0.617	0.613	0.609	0.607	0.606	0.605	0.605	0.603	0.602	0.601
1.8	0.638	0.627	0.620	0.616	0.612	0.609	0.607	0.606	0.605	0.605	0.603	0.602	0.602
2.0	0.637	0.626	0.619	0.615	0.612	0.608	0.606	0.606	0.605	0.605	0.604	0.602	0.602
2.5	0.634	0.624	0.617	0.613	0.610	0.607	0.606	0.606	0.605	0.605	0.604	0.603	0.602
3.0	0.632	0.622	0.616	0.612	0.609	0.607	0.606	0.606	0.605	0.605	0.604	0.603	0.603
3.5	0.630	0.621	0.615	0.611	0.609	0.607	0.606	0.605	0.605	0.605	0.604	0.603	0.602
4.0	0.628	0.619	0.614	0.610	0.608	0.606	0.606	0.605	0.605	0.605	0.603	0.603	0.602
5.0	0.626	0.617	0.613	0.610	0.607	0.606	0.605	0.605	0.604	0.604	0.603	0.602	0.602
6.0	0.623	0.616	0.612	0.609	0.607	0.605	0.605	0.605	0.604	0.604	0.603	0.602	0.602
7.0	0.621	0.615	0.611	0.608	0.607	0.605	0.605	0.604	0.604	0.604	0.603	0.602	0.602
8.0	0.619	0.613	0.610	0.608	0.606	0.605	0.604	0.604	0.604	0.603	0.603	0.602	0.602
9.0	0.618	0.612	0.609	0.607	0.606	0.604	0.604	0.604	0.603	0.603	0.602	0.602	0.601
10.0	0.616	0.611	0.608	0.606	0.605	0.604	0.604	0.603	0.603	0.603	0.602	0.602	0.601
20.0	0.606	0.605	0.604	0.603	0.602	0.602	0.602	0.602	0.602	0.601	0.601	0.601	0.600
50.0	0.602	0.601	0.601	0.601	0.601	0.600	0.600	0.600	0.600	0.600	0.599	0.599	0.599
100.0	0.599	0.598	0.598	0.598	0.598	0.598	0.598	0.598	0.598	0.598	0.598	0.598	0.598

[1] From Hamilton Smith's "Hydraulics."

TABLE 9.—DISCHARGE PER FOOT OF LENGTH OVER RECTANGULAR NOTCH WEIRS

(From Slocum's "Elements of Hydraulics")

Discharge over sharp crested, vertical, rectangular notch weirs in cubic feet per second per foot of length. Computed from Eq. (41), Par. 66

$$Q = 3.3bh^{3/2} \text{ for } b = 1 \text{ ft.}$$

Depth on crest in feet	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.000	0.003	0.009	0.017	0.026	0.037	0.049	0.061	0.075	0.089
0.1	0.104	0.120	0.137	0.155	0.173	0.192	0.211	0.231	0.252	0.273
0.2	0.295	0.317	0.341	0.364	0.388	0.413	0.438	0.463	0.489	0.515
0.3	0.542	0.570	0.597	0.626	0.654	0.683	0.713	0.743	0.773	0.804
0.4	0.835	0.866	0.898	0.931	0.963	0.996	1.030	1.063	1.098	1.132
0.5	1.167	1.202	1.238	1.273	1.309	1.346	1.383	1.420	1.458	1.496
0.6	1.538	1.572	1.611	1.650	1.690	1.729	1.769	1.810	1.850	1.892
0.7	1.933	1.974	2.016	2.058	2.101	2.143	2.187	2.230	2.273	2.317
0.8	2.361	2.406	2.450	2.495	2.541	2.586	2.632	2.678	2.724	2.768
0.9	2.818	2.865	2.912	2.960	3.008	3.055	3.104	3.152	3.202	3.251
1.0	3.300	3.350	3.399	3.449	3.501	3.551	3.600	3.653	3.703	3.755
1.1	3.808	3.858	3.911	3.963	4.016	4.069	4.122	4.178	4.231	4.283
1.2	4.340	4.392	4.448	4.501	4.557	4.613	4.666	4.722	4.778	4.834
1.3	4.891	4.947	5.006	5.062	5.118	5.178	5.234	5.293	5.349	5.409
1.4	5.468	5.524	5.584	5.643	5.702	5.762	5.821	5.881	5.940	6.003
1.5	6.062	6.125	6.184	6.247	6.306	6.369	6.428	6.491	6.554	6.617
1.6	6.679	6.742	6.805	6.867	6.930	6.993	7.059	7.121	7.187	7.250
1.7	7.316	7.379	7.445	7.508	7.573	7.639	7.706	7.772	7.838	7.904
1.8	7.970	8.036	8.102	8.171	8.237	8.303	8.372	8.438	8.507	8.573
1.9	8.643	8.712	8.778	8.847	8.917	8.986	9.055	9.125	9.194	9.263
2.0	9.332	9.405	9.474	9.544	9.616	9.686	9.758	9.827	9.900	9.969
2.1	10.042	10.115	10.187	10.260	10.332	10.405	10.478	10.550	10.623	10.695
2.2	10.768	10.841	10.916	10.989	11.065	11.138	11.213	11.286	11.362	11.435
2.3	11.510	11.586	11.662	11.738	11.814	11.887	11.966	12.042	12.118	12.194
2.4	12.269	12.345	12.425	12.500	12.576	12.656	12.731	12.811	12.890	12.966
2.5	12.935	13.124	13.200	13.279	13.358	13.438	13.517	13.596	13.675	13.754
2.6	13.834	13.916	13.995	14.075	14.154	14.236	14.315	14.398	14.477	14.560
2.7	14.642	14.721	14.804	14.886	14.969	15.048	15.131	15.213	15.296	15.378
2.8	15.461	15.543	15.629	15.711	15.794	15.876	15.962	16.045	16.130	16.213
2.9	16.299	16.381	16.467	16.550	16.635	16.721	16.807	16.889	16.975	17.061
3.0	17.147	17.233	17.318	17.404	17.490	17.579	17.665	17.751	17.837	17.926
3.1	18.011	18.101	18.186	18.275	18.361	18.450	18.536	18.625	18.714	18.803
3.2	18.889	18.978	19.067	19.157	19.246	19.335	19.424	19.513	19.602	19.694
3.3	19.784	19.873	19.962	20.054	20.143	20.236	20.325	20.414	20.506	20.599
3.4	20.688	20.780	20.873	20.962	21.054	21.146	21.239	21.331	21.424	21.516
3.5	21.608	21.701	21.793	21.886	21.978	22.074	22.166	22.259	22.354	22.447
3.6	22.542	22.635	22.730	22.823	22.919	23.011	23.107	23.202	23.295	23.390
3.7	23.486	23.582	23.678	23.773	23.869	23.965	24.060	24.156	24.252	24.347
3.8	24.446	24.542	24.638	24.734	24.833	24.928	25.027	25.123	25.222	25.318
3.9	25.417	25.516	25.611	25.710	25.809	25.905	26.004	26.103	26.202	26.301
4.0	26.400	26.499	26.598	26.697	26.796	26.895	26.997	27.096	27.195	27.298

TABLE 10.[1]—COEFFICIENTS OF PIPE FRICTION

(From Slocum's "Elements of Hydraulics")

Value of the friction coefficient f, in the formula

$$h = f\frac{l}{d} \cdot \frac{v^2}{2g}$$

Computed from the exponential formulas of Thrupp, Tutton and Unwin

Material	Diameter in inches	Velocity of flow in feet per second				
		2	4	6	8	10
Lead pipe	1	0.032	0.026	0 024	0.022	0 021
	2	0.030	0.025	0.023	0.021	0 020
	3	0.029	0.024	0.022	0.020	0 019
	4	0.028	0.023	0.021	0.020	0.019
Wood pipe	6	0.034	0.033	0.032	0.032	
	12	0.027	0.027	0.026	0.026	
	18	0.024	0.024	0.023	0.023	
	24	0 022	0.022	0.021	0.021	
	36	0.020	0.019	0.019	0.019	
	48	0.018	0.018	0.017	0.017	
Asphalted pipe	6	0.026	0.023	0.022	0.021	0 020
	9	0.025	0.022	0.021	0.020	0.019
	12	0.024	0.021	0.020	0.019	0.019
	18	0.023	0.020	0.019	0.018	0.018
	24	0.022	0.020	0.018	0.017	0.017
	36	0.021	0.019	0.017	0.017	0.016
	48	0.020	0.018	0.017	0.016	0.015
Bare wrought-iron pipe	3	0.024	0.021	0 019	0.018	0.017
	6	0.022	0.019	0.017	0.016	0.016
	12	0.019	0 017	0.015	0.014	0.014
	24	0.017	0.015	0.014	0.013	0.012
	36	0.016	0.014	0.013	0.012	0.011
	48	0.015	0.013	0.012	0.011	0.011
	60	0.015	0.013	0.012	0.011	0.010
Riveted wrought-iron or steel pipe	12	0.025	0.022	0.021	0.020	0.019
	24	0 020	0.018	0.017	0.016	0.016
	36	0.017	0.016	0.015	0.014	0.014
	48	0.016	0.014	0.014	0.013	0.013
	60	0.015	0.013	0.013	0.012	0.012
	72	0.014	0.013	0.012	0.011	0.011
New cast-iron pipe	3	0.028	0.026	0.025	0.025	
	6	0.024	0.022	0.022	0.021	
	9	0.021	0 020	0.020	0.019	
	12	0.020	0.019	0.018	0.018	
	18	0.018	0.017	0.017	0.016	
	24	0.017	0.016	0.016	0.015	
	36	0.015	0.015	0.014	0.014	
Old cast-iron pipe	3	0.059	0.058	0.058	0.058	
	6	0.050	0.050	0.050	0.049	
	9	0.046	0.045	0.045	0.044	
	12	0.043	0.042	0.042	0.042	
	18	0.039	0.039	0.038	0.038	
	24	0.037	0.036	0.036	0.036	
	36	0.033	0.033	0.033	0.032	

[1] Compiled from data in GIBSON'S "Hydraulics."

TABLE 11.—KUTTER'S VALUES OF CHEZY'S COEFFICIENT

(From Slocum's "Elements of Hydraulics")

Values of the coefficient C in Chezy's formula $v = C\sqrt{rs}$ according to Kutter's formula (Eq. (94) Par. 140).

$$C = \frac{41.65 + \frac{0.00281}{s} + \frac{1.811}{n}}{1 + \left(41.65 + \frac{0.00281}{s}\right)\frac{n}{\sqrt{r}}}$$

Slope, s	Coefficient of roughness, n	Hydraulic radius r, in feet														
		0.1	0.2	0.4	0.6	0.8	1	1.5	2	3	4	6	8	10	15	20
s = 0.000025 = 1 in 40,000 = 0.132 ft. per mile	0.009	65	87	111	127	138	148	166	179	197	209	226	238	246	262	271
	0.010	57	75	97	112	122	131	148	160	177	188	206	216	225	240	249
	0.011	50	67	87	100	109	118	133	144	160	172	188	199	207	222	231
	0.012	44	59	78	90	99	106	121	131	147	158	174	184	192	206	215
	0.013	40	53	70	81	90	97	111	121	135	146	161	171	179	193	202
	0.017	28	38	51	60	66	72	83	91	103	113	126	135	142	155	164
	0.020	23	31	42	49	55	60	69	77	88	96	108	117	124	136	144
	0.025	17	24	32	38	43	47	55	61	70	78	88	96	102	114	121
	0.030	14	19	26	31	35	38	45	50	59	65	74	82	87	98	106
	0.035	12	16	22	26	30	32	38	43	50	56	64	71	76	86	94
s = 0.00005 = 1 in 20,000 = 0.264 ft. per mile	0.009	78	100	124	139	150	158	173	184	198	207	220	228	234	244	250
	0.010	67	87	109	122	133	140	154	164	178	187	199	206	212	220	228
	0.011	59	77	97	109	119	126	139	148	161	170	182	189	195	205	211
	0.012	52	68	88	98	107	114	126	135	148	156	168	175	181	189	196
	0.013	47	62	79	90	98	104	116	124	136	145	156	163	169	179	184
	0.017	33	44	57	65	71	77	87	94	104	111	122	129	134	142	149
	0.020	26	35	46	53	59	64	72	79	88	95	105	111	116	125	131
	0.025	20	26	35	41	46	49	57	62	71	77	85	91	96	104	110
	0.030	16	21	28	33	37	40	47	51	59	64	72	78	82	90	96
	0.035	13	18	24	28	31	34	40	44	50	56	63	68	72	79	85
s = 0.0001 = 1 in 10,000 = 0.528 ft. per mile	0.009	90	112	136	149	158	166	178	187	198	206	215	221	226	233	237
	0.010	78	98	119	131	140	147	159	168	178	186	195	201	205	212	216
	0.011	68	86	106	118	126	132	144	151	162	169	178	184	188	195	200
	0.012	60	76	95	105	114	120	130	138	149	155	164	170	174	181	185
	0.013	54	69	86	96	103	109	120	127	137	143	152	158	162	169	173
	0.017	37	48	62	70	76	81	89	96	104	111	119	124	128	135	139
	0.020	30	39	50	57	63	67	75	81	89	94	102	107	111	118	122
	0.025	22	29	38	44	48	52	59	64	71	76	84	88	92	98	102
	0.030	17	23	31	35	39	42	48	53	59	64	71	75	78	85	89
	0.035	14	19	25	30	33	35	41	45	51	55	61	66	69	75	79

Table 12.—Sand Ejectors and Flow of the Water and Sand in Discharge Piping[1]

Pounds pressure feed water	Diameter jet, inches	Best diameter for throat, inches	Per cent. sand in discharge by volume	Cu. yd sand per hour	Pressure of discharge in feet	Friction in feet per 1,000 in discharge piping			
						2½″	3″	4″	5″
60	0 5	0 87	20	5 0	28.	150	140		
	0 5	1 01	25	7 2	20	176	150		
	0 5	1 21	30	10 0	14	206	168		
	0 6	1 04	20	7 2	28	178	124	90	
	0 6	1 21	25	10 3	20	222	144	110	
	0.6	1 46	30	14 3	14	275	170	120	
	0 7	1 06	15	6 5	39	200	114	70	
	0 7	1 21	20	9.7	28	250	140	88	
	0 7	1 41	25	14.0	20	325	175	102	
	0 8	1 21	15	8.5	39	298	145	71	
	0 8	1 39	20	12 7	28	370	180	89	73
	0 8	1 61	25	18 4	20	480	240	109	82
	0 9	1.20	10	6 5	52	350	160	61	42
	0 9	1 37	15	10.8	39	435	202	80	56
	0.9	1 56	20	16 1	28	540	250	102	71
80	0 5	0 87	20	5.7	38	154	130		
	0 5	1 01	25	8.3	27	186	145		
	0.5	1 21	30	11.5	18	225	160		
	0 6	1 04	20	8 3	38	208	128	90	
	0.6	1.21	25	11.9	27	260	153	107	
	0.6	1.46	30	16 5	18	320	187	117	
	0 7	1.06	15	7.5	52	245	127	70	
	0 7	1 21	20	11.2	38	305	158	86	
	0.7	1 41	25	16.2	27	400	204	104	71
	0 8	1 21	15	9.8	52	370	176	76	56
	0 8	1 39	20	14 7	38	465	222	95	71
	0 8	1 61	25	21 2	27	600	290	115	84
	0 9	1 20	10	7.5	70	450	200	70	43
	0.9	1 36	15	12 4	52	550	248	91	58
	0.9	1.56	20	18.6	38	680	315	114	73
100	0 5	0 87	20	6 4	47	162	126		
	0 5	1 01	25	9 2	34	200	143		
	0 5	1.21	30	12 9	23	250	164	120	
	0 6	1 04	20	9 2	47	230	133	87	
	0 6	1 21	25	13 3	34	300	165	103	
	0.6	1 46	30	18.5	23	380	210	118	105
	0 7	1 06	15	8.4	65	292	144	71	
	0 7	1 21	20	12.6	47	360	180	88	71
	0.7	1 41	25	18.1	34	470	235	108	83
	0 8	1 21	15	11 0	65	450	208	82	56
	0.8	1 39	20	16 4	47	550	260	104	71
	0 8	1.61	25	23.7	34	...	340	130	86
	0 9	1.20	10	8 3	87	...	240	78	45
	0 9	1.36	15	13 9	65	...	300	100	60
	0.9	1.56	20	20.8	47	...	370	128	76
150	0.5	0.87	20	7 8	71	195	124	90	
	0 5	1.01	25	11 4	51	240	150	110	
	0.5	1.21	30	15 8	35	305	180	118	
	0.6	1.04	20	11.3	71	310	160	86	75
	0.6	1.21	25	16 3	51	400	205	104	83
	0 6	1.46	30	22 7	35	520	260	125	100
	0.7	1 06	15	10 3	98	410	190	78	56
	0.7	1.21	20	15.4	71	500	236	98	71
	0.7	1.41	25	22.2	51	640	310	120	85
	0.8	1.21	15	13.5	98	...	285	99	59
	0 8	1.39	20	20 0	71	...	350	123	75
	0.8	1.61	25	29.0	51	...	450	160	93
	0.9	1.20	10	10.2	131	...	335	100	50
	0 9	1.36	15	17.0	98	...	410	129	67
	0.9	1.56	20	25.5	71	...	510	163	85

[1] Taken from American Civil Engineers' Pocket Book by permission of author, Mr. Allen Hazen and of publishers, John Wiley & Sons, Inc.

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